HIGHWAY RESEARCH BOARD
Special Report 87

HIGHWAY
CAPACITY
MANUAL
1965

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Errata

The 1965 *Highway Capacity Manual* was reprinted for the eighth time in September 1978. The number and the date of each of the eight printings are shown on the reverse side of the title page. This errata is being issued so that those who have earlier printings, particularly the first, will be aware of corrections that have subsequently been made. The number of the printing to which the correction applies is given in the left column. All corrections given here have been incorporated in the eighth printing.

The Transportation Research Board expresses its appreciation to members of the staff at the Office of Traffic Operations, Federal Highway Administration, which has maintained the list of corrections to the *Highway Capacity Manual.*

Printing	Correction
1,2,3, 4,5,6	Page xiii, line 12 After **Lateral,** delete **Consideration** and add **Clearance**
1	Pages 49 and 50 Interchange line drawings but not titles of Figure 3.26 and Figure 3.28. (Suggestion: Clip drawings, flip over, and tape in place.)
1	Page 54, Figure 3.31, title After **1957.** add **It should be noted that in this study the median lane was designated as Lane 1; the shoulder lane, as Lane 3.**
1	Page 62, column 2, line 22 After **highway, the** delete **space mean** and insert **operating**
1	Page 66, Figure 3.44, title After **under,** delete **interrupted** and insert **uninterrupted**
1	Page 90, column 2, line 1 After **of** delete **roadway** and insert **lane**
1	Page 102, column 2, line 13 After **car** delete **equivalency factors** and insert **equivalents**
1,2,3, 4,5,6	Page 102, column 2, line 21 After **determine the** delete **equivalency factor** and insert **passenger car equivalents**
1	Page 102, column 2, line 35 Before **contains** delete **Table 10.10** and insert **Table 10.12**
1	Page 104, column 1, line 45 After **give** delete **equivalency factors** and insert **equivalents**
1	Page 126, column 2, lines 15 and 19 After **green** add **per lane**
1,2	Page 128, column 2, line 13 Delete last three sentences in paragraph beginning **The safest** and ending **the intersection.** and insert **Standard values for yellow interval have not been defined in the Manual; how- ever, they are variable within limits which are established in the Manual on Uniform Traffic Control Devices for Streets and Highways. When additional time is required for the motorist to clear a wide intersection, an all-red interval may be provided immedi- ately following the yellow interval. If total time of the yellow and all-red periods is held to the minimum necessary for safe operation, intersection capacity will be improved.**
1,2,3, 4,5,6	Page 138, column 2, lines 11 and 12 Delete **used. It is recommended that a 3-sec yellow interval normally be allowed** and insert **used, and the length should be as recommended in the Manual on Uniform Traffic Control Devices.**
1	Page 148, column 2, lines 11 and 15 After **with parking** add **both sides**

Printing	Correction
1	Page 149, column 1, lines 11 and 37 After **parking, at** delete **capacity** and insert **level of service C**
1	Page 153, column 2, line 13 After **1,255x55/** delete **70** and insert **90**
1,2,3, 4,5,6	Page 153, column 2, line 29 Before **volume,** delete **Through** and insert **Total approach**
1	Page 155, column 1, line 20 After **Adjustment for** delete **3%** and insert **4%**
1,2	Page 155, column 1, line 21 After = delete **1.035** and insert **1.030**
1	Page 155, column 1, line 26 After **0.72x** delete **1.035** and insert **1.030**
1	Page 155, column 1, line 27 Before **vph** delete **1,220** and insert **1,215**
1,2	Page 169, column 1, line 13 Delete **For the better levels of service the** and insert **The**
1,2	Page 169, column 2, line 13 Delete paragraph beginning **In determining** and ending **grades, etc.** and insert **These values, after adjustment for prevailing roadway and traffic conditions, should be used as a check against the computed SV. The lower of the two values should be used in determining N. Normally this procedure will affect values at levels D and E.**
1	Page 180, column 1, line 4 After = delete **3** and insert **2**
1	Page 180, column 1, line 6 After = delete **0.89** and insert **0.94**
1	Page 180, column 1, line 7 After **410)/** delete **0.89** and insert **0.94**
1	Page 180, column 1, line 8 Before **pcph** delete **1,020** and insert **970**
1	Page 180, column 1, line 17 After **is** delete **1,400** and insert **1,300**
1	Page 180, column 2, line 4 After **1,375** delete **(0.89) = 1,225** and insert **(0.94) = 1,290**
1	Page 180, column 2, line 9, denominator Delete **1,225** and insert **1,290**

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Printing	Correction
1	Page 180, column 2, line 16 After **is** delete **550** and insert **500**
1	Page 180, column 2, line 10 After = delete **3.5** and insert **3.3**
1	Page 180, column 2, line 23 After **1,375** delete **(0.89) = 1,225** and insert **(0.94) = 1,290**
1	Page 180, column 2, line 27 Delete **1,225 = 1.4** and insert **1,290 = 1.3**
1	Page 180, column 2, line 30 After **2,700/** delete **1,225 = 2.2** and insert **1,290 = 2.1**
1	Page 180, column 2, line 40 After **of** delete **1,400** and insert **1,300**
1	Page 180, column 2, line 41 After **of** delete **550** and insert **500**
1	Page 187, column 1, paragraph 2, line 2 Delete entire line and insert **tions and influences may be applied to all**
1	Page 203, Table 8.2, column 2, row 4 Delete **Fig. 8.5** and insert **Fig. 8.4**
1,2	Page 203, Table 8.2, column 6, row 7 After **(Table 8.3 and** delete **Fig. 24b)** and insert **Fig. 8.24a)**
1,2,3, 4,5,6	Page 208, Figure 8.7, Conditions for Use, line 8 After **usage on** delete **p. 220** and insert **p. 222**
1,2	Page 209, Figure 8.7, title After **junction,** delete **6-lane freeway, with adjacent off-ramps both upstream and down-stream of stream off-ramp** and insert **4-lane freeway, with auxiliary lane between on-ramp and adjacent downstream off-ramp**
1,2,3, 4,5,6	Page 212, Figure 8.10 On drawing in box on right side, label freeway through volume V_t, upstream on-ramp volume V_u, and off-ramp volume V_r
1,2	Page 230, column 2, line 10 After = delete **273** and insert **372**
1,2	Page 231, column 1, line 29 After **Figure** delete **8.23** and insert **8.24**
1,2	Page 231, column 2, line 20 After **Figure** delete **8.23b** and insert **8.24b**

Printing	Correction
1,2	Page 231, column 2, line 25 After **Figure** delete **8.23b** and insert **8.24b**
1	Page 246, column 1, lines 37 and 38 After **Three** delete **(as Fig. 3.23 and as Figs. 3.35, 3.38, and 3.41** and insert **(as Fig. 3.26 and as Figs. 3.38, 3.41, and 3.44**
.1	Page 252, Table 9.1, columns 4, 5, 6, 7, and 8, rows 3 and 4 Delete **xPHF** and insert **(PHF)**
1,2	Page 256, column 1, line 9 Delete entire line and insert **the passenger car equivalents from Table 9.3a.**
1	Page 264, column 1, line 15 After **in** delete **Figure 3.35** and insert **Figure 3.38**
1,2,3, 4,5,6	Page 265, column 2, lines 28 and 29 After **volume** delete **(total for one direction)** and insert **(mixed vehicles per hour, in one direction)**
1,2,3, 4,5,6	Page 265, column 2, lines 30, 31, and 32 After **lanes** delete **(mixed vehicles per hour, in one direction)**
1,2,3, 4,5,6	Page 272, column 1, line 33 Before **mph** delete **55** and insert **50**
1,2,3, 4,5,6	Page 272, column 1, line 35 Before **indicates** delete **0.60** and insert **0.78**
1,2,3, 4,5,6	Page 272, column 1, line 36 Before **mph** delete **55** and insert **50**
1,2,3, 4,5,6	Page 272, column 1, line 39 After **=** delete **C** and insert **D**
1,2,3, 4,5,6,7	Page 273, column 1, line 4 Before **at-grade** insert **no significant**
1	Page 282, column 2, lines 42 and 43 After **as** delete **Figure 3.24 and Figures 3.36 and 3.39** and insert **Figure 3.27 and Figures 3.39 and 3.42**
1	Page 293, column 2, line 14 After **in** delete **Figure 3.36** and insert **Figure 3.39**
1,2,3, 4,5,6	Page 295, column 2, lines 22 and 33 Before **conditions** delete **ideal** and insert **prevailing**
1	Page 299, column 2, line 38 After **in** delete **Figure 3.25** and insert **Figure 3.28**

Printing	Correction

1   Page 299, column 2, line 44
After **in** delete **Figures 3.37 and 3.40** and insert **Figures 3.40 and 3.43**

1   Page 300, column 1, line 18
After **reach** delete **30** and insert **20**

1,2,3,
4,5,6   Page 308, column 1, line 1
After **rate** delete **truck factors** and insert **equivalents**

1,2,3,
4,5,6   Page 315, column 2, line 26
Before **for** insert **from Table 10.7**

1,2,3,
4,5,6,7   Page 315, column 2, line 32
Before = delete T_c and insert T_L

1,2,3,
4,5,6,7   Page 315, column 2, line 33
Before = delete B_c and insert B_L

1,2,3,
4,5,6   Page 316, drawing, section 2
Before **CURVE** delete 3° and insert $7\frac{1}{2}^\circ$

1,2,3,
4,5,6.   Page 316, drawing, section 4
Before **CURVE** delete 4° and insert 5°

1,2,3,
4,5,6   Page 317, column 2
Delete lines 1 through 24 and insert the following:

Base volume = 2,000 $W_L T_L$,

where:
    W_L, from Table 10.8, for 11-ft equivalent
       lanes and adequate clearance, at level
       D = 0.87.
    T_L is derived as follows, using a weighted
       E_T value:
       $E_{T(up)}$: From Table 10.10, for level D,
         given 3% trucks on 3% grade 1½ mi
         long, $E_{T(up)} = 26$.
       $E_{T(down)}$: Given as $E_{T(down)} = 10$.

$$E_{T(overall)} = \frac{E_{T(up)}P_{T(up)} + E_{T(down)}P_{T(down)}}{P_{T(total)}}$$

$$= \frac{26 \times 3 + 10 \times 5}{8} = 16$$

From Table 10.12, for 8% trucks and
    $E_T = 16$, $T_L = 0.45$.

Base volume = 2,000 × 0.87 × 0.45 = 783 vph.
    v/c ratio = 500/783 = 0.64
From Table 10.7 or Fig. 10.2a, for AHS =
    67 mph and 45% passing sight distance,
    service is in level D (close to level C);
    assumption was correct.

| 1,2,3, | Page 323, Table 10.13, footnote a |
| 4,5,6 | Delete **Operating speed** and insert **Average overall travel speed** |

ERRATA

HRB Special Report 87, "Highway Capacity Manual - 1965."

In HRB Special Report 87, "Highway Capacity Manual - 1965," first printing (March 1966)*, the following changes should be noted:

Pages 49 and 50. Interchange Figs. 3.26 and 3.28. Captions are correct as they are.

Page 54, Figure 3.31. To caption, add "It should be noted that in this study the median lane was designated as Lane 1; the shoulder lane, as Lane 3."

Page 62, Col. 2, line 22. Change "space mean" to "operating."

Page 66, Figure 3.44. In caption, change "interrupted" to "uninterrupted."

Page 90, Col. 2, line 1. Change "roadway" to "lane."

Page 102, Col. 2, lines 11 and 13-14. Change "passenger car equivalency factors" to "passenger car equivalents."

    Col. 2, line 35. Change "Table 10.10" to "Table 10.12."

Page 104, Col. 1, lines 45-46. Change "equivalency factors" to "equivalents."

Page 126, Col. 2, lines 15 and 19. After "green" add "per lane."

Page 148, Col. 2, lines 11 and 15. To "with parking" add "both sides."

Page 149, Col. 1, lines 11 and 37-38. Change "capacity" to "level of service C."

Page 153, Col. 2, line 13. Change "70" to "90."

Page 155, Col. 1, line 20. Change "3%" to "4%."
               line 26. Change "1.035" to "1.030."
               line 27. Change "1,220" to "1,215."

Page 180, Col. 1, line 4. Change "3" to "2."
          Col. 1, lines 6 and 7. Change "0.89" to "0.94." (two places)
          Col. 1, line 8. Change "1,020" to "970."
          Col. 1, line 17. Change "1,400" to "1,300."
          Col. 2, line 4. Change "(0.89)=1,225" to "(0.94)=1,290."
          Col. 2, line 9. Change "1,225" to "1,290."
          Col. 2, line 10. Change "3.5" to "3.3."
          Col. 2, line 16. Change "550" to "500."
          Col. 2, line 23. Change "(0.89)=1,225" to "(0.94)=1,290."
          Col. 2, line 27. Change "1,225 = 1.4" to "1,290 = 1.3."
          Col. 2, line 30. Change "1,225 = 2.2" to "1,290 = 2.1."
          Col. 2, line 40. Change "1,400 to "1,300."
          Col. 2, line 41. Change "550" to "500."

** Page 187, Col. 1, 2nd Par. Delete line 2 (duplicate of line 4) and replace with "tions and influences may be applied to all ------------."

Page 203, Table 8.2. Under 4-Lane Freeway On-Ramp, the entry for the fourth arrangement should read "Fig. 8.5." Entry for Off-Ramp is shown correctly as "Fig. 8.4."

Page 246, Col. 1, lines 37-38. Should read "(as Fig. 3.26 and as Figs. 3.38, 3.41, and 3.44 ----)."

Page 252, Table 9.1, Level of Service C and D. In seven entries under Service Volume/Capacity, parentheses indicate multiplication of coefficient and PHF.

Page 264, Col. 1, line 15. Should read "Fig. 3.38."

Page 282, Col. 2, lines 42 and 43. Should read "as Fig. 3.27 and Figs. 3.39 and 3.42 ----"

Page 293, Col. 2, line 14. Should read "Fig. 3.39."

Page 299, Col. 2, line 39. Should read "Fig. 3.28."
          line 44. Should read "Figs. 3.40 and 3.43."

Page 300, Col. 1, line 18. Change "30" to "20."

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* In the second printing (October 1966) the changes listed have been made.

** This correction was made in 4,000 copies of the 10,000-copy first printing.

HIGHWAY RESEARCH BOARD

Special Report 87

HIGHWAY CAPACITY

MANUAL

1965

Subject Classification

22 Highway Design
53 Traffic Control and Operations
54 Traffic Flow
55 Traffic Measurements

FOREWORD

A rational and practical method for the determination of highway capacity is essential in the sound economic and functional design of new highways and in the adaptation to present or future needs of the many existing roads and streets which must continue in use for extended periods of time. Basically, the subject concerns the effectiveness of various facilities in the service of traffic, and involves the many elements of highway design, vehicle and driver performance, and traffic control.

Highway capacity has been the subject of continuing study over a long period of years, and extensive literature in the field has resulted. In recent years, due to the development of refined traffic study methods, instrumentation, and equipment, substantial amounts of reliable field data have been gathered.

The cooperative efforts of the Highway Research Board's Committee TO-4 on Highway Capacity, the Bureau of Public Roads, many state and city traffic engineering organizations, universities, and consultants, have resulted in the assembly and analysis of large amounts of these data, and their consolidation with other reported research results.

As in the original 1950 edition of this manual, the traffic-carrying capabilities of all common types of highways and elements thereof are discussed. Because freeways are now in widespread use, new emphasis is placed on freeways and their appurtenances, including ramps and weaving sections. However, other types of highways, including ordinary rural multilane and two-lane roads, urban arterials, and downtown streets, continue to receive complete coverage, as do at-grade intersections.

This manual, like the original edition, is primarily a practical guide. It permits determination of the capacity, service volume, or level of service which will be provided by either a new highway design or an existing highway, under specified conditions. Alternately, given a certain traffic demand, the design necessary to accommodate that demand at a given level of service can be determined.

Preparation of this manual was the responsibility of the Highway Research Board Committee TO-4 on Highway Capacity, chaired by the late O. K. Normann, Deputy Director for Research, Office of Research and Development, Bureau of Public Roads, until his death in 1964, then by Carl C. Saal, his successor as Deputy Director for Research. The members of this committee are listed on the preceding page, together with their affiliations. The committee was divided into eight subcommittees, each of which was given the responsibility for a major topic area. Each member served on at least two subcommittees.

Work on the assembly of new material for this revision began, in the Bureau of Public Roads, as early as 1954, when a comprehensive nationwide intersection study program was initiated. In 1957, the Committee began detailed planning for a new edition. Progress was gradual until 1963, when a five-man task group was assigned by the Bureau of Public Roads to full-time work on the manual for several months. This group, from four BPR field offices, consisted of:

John B. Kemp, *Chairman,* then BPR Division Engineer, North Dakota (now Division Engineer, Kentucky)

Steiner M. Silence, Highway Engineer (Traffic), Region 4 Office, Homewood, Ill.

Howard C. Hanna, Urban Transportation Planning Engineer, Region 4 Office, Homewood, Ill.

I. Chester Jenkins, Geometric Design Engineer, Region 3 Office, Atlanta, Ga.

Robert E. Johnson, Planning and Research Engineer, Region 1 Office, Delmar, N. Y

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FIGURES   *Page*

TABLES   *Page*

PHOTO CREDITS

The following organizations contributed the photographs appearing on the pages listed:

American Road Builders' Association—193; American Transit Association—21, 344; Automotive Safety Foundation—77 (left), 95; Bureau of Public Roads—4, 7, 9, 17, 18, 19, 31, 43, 113, 119, 123, 125, 169, 171, 172, 174, 175, 191, 192, 251, 269, 278, 283, 293, 337, 339; California Division of Highways—1, 5, 6, 107, 120, 189, 195, 347; Cleveland (Ohio) Transit System—333, 342, 343, 345; District of Columbia Department of Highways and Traffic—13; Michigan State Highway Department—199; Montana State Highway Department—93; New Hampshire Department of Highways—247; New York State Department of Public Works—77 (right), 84, 85, 90, 92, 115, 157, 174, 254, 297, 300; North Carolina State Highway Commission—11, 36, 51, 71, 110; The Port of New York Authority—71, 82, 83, 185, 242; Rhode Island State Highway Department—328; South Carolina Highway Department—91; South Dakota Department of Highways—33; Texas Highway Department—249.

CHAPTER ONE

INTRODUCTION

The ability to accommodate vehicular traf-
fic is a primary consideration in the planning,
design, and operation of streets and high-
ways. Highway capacity is, very broadly,
a measure of the effectiveness of various
highways in accommodating traffic, and its
application requires both a general knowl-
edge of traffic behavior and specific knowl-
edge of traffic volumes that can be accom-
modated under a variety of roadway con-
figurations and operating conditions. A
rational and practical method for determin-

ing highway capacities is essential for sound
economic and functional utilization of the
highway transportation system.

Specifically, *capacity* is here defined as
the maximum number of vehicles per unit
of time that can be handled by a particular
roadway component under the prevailing
conditions.

Where appropriately in context, however,
the entire broad subject area will be referred
to simply as "capacity" in this manual.

It is of little value to know the quantita-

tive measure alone, without knowing the prevailing conditions. Similarly, the overall traffic-carrying capabilities of a roadway cannot be treated without reference to other important considerations, such as the quality of service provided and the duration of the time period considered, because capacity is only one of many service levels at which the roadway may operate. Different roadways of a given type may have different capacities, depending on both the physical characteristics and the operating conditions.

The original (1950) edition of this manual defined three levels of roadway capacity —basic capacity, possible capacity and practical capacity. Basic capacity was defined as "the maximum number of passenger cars that can pass a point on a lane or roadway during one hour under the most nearly ideal roadway and traffic conditions which can possibly be attained." Possible capacity was the "maximum number of vehicles that can pass a given point on a lane or roadway during one hour, under prevailing roadway and traffic conditions," whereas practical capacity was a lower volume chosen "without the traffic density being so great as to cause unreasonable delay, hazard, or restriction to the drivers' freedom to maneuver under prevailing roadway and traffic conditions."

That edition of the manual "considered it to be of prime importance to relate traffic volumes accurately to operating conditions so that individual agencies with a thorough knowledge of the specific conditions could decide on the most practical volumes to expect a facility to handle." Though thus recognizing that "practical" capacity varies considerably on the basis of a subjective determination of the quality of service to be provided, the manual did suggest values for practical capacities under various specific conditions based on the normal desires of typical drivers.

The present Committee has elected, in this new edition, to define only a single "capacity" for each type of highway. "Capacity," as now defined, is the same as "possible capacity" in the 1950 edition. The former "basic capacity" has now been replaced with the phrase "capacity under ideal conditions." Thus, "capacity" represents a positive quantity—the maximum traffic a given roadway can handle. Several "service volumes" now replace the earlier "practical capacities," representing any of several specific volumes related to a group of desirable operating conditions collectively termed "level of service." Level of service is a qualitative measure and the actual value used should be appropriate for the highway.

The time period considered should be defined in evaluating capacity determinations. For short time periods (one hour or less), capacity is a maximum sustained rate of flow for the specified time period. When considering longer time periods, such as a day or a year, capacity also depends on travel desires which create hourly, daily, and seasonal fluctuations in an average traffic volume which will result in full utilization of the roadway for only a percentage of the total time period, when demand is the heaviest.

The original manual, which this edition replaces, provided methods for measuring capacity and relating it to speed and spacing criteria. The procedures were based mainly on the empirical approach where relationships between two or more variables are developed from field observations. In this new edition, the same approach has been used, to a large degree. Many recent studies have relied more on the theoretical or experimental methods of mathematically expressing traffic flow. The results of these methods, which represent a new approach to the complexities of traffic flow, very likely will ultimately provide better answers to many highway capacity problems; they are utilized in this manual to the extent possible. At the present time, however, consideration of many of the elements of the real situation remain lacking in certain of the theoretical approaches, and reliance still must be placed largely on empirical data and statistical analyses, coupled with good judgment.

The purpose of this manual, then, is to provide a condensed and authoritative source of the present empirical and theoretical information on highway capacity. By providing a standard set of terminology and methods of measurement and analysis, the

manual will aid in the study and evaluation of existing facilities. By its description of traffic behavior found in field studies, it will aid in predicting the capacity and level of service of proposed improvements. From the initial planning stage to the correction of operating problems, this knowledge is necessary to establish expected capacity values for consideration in engineering and economic comparisons.

The information given in this manual has been selected to represent typical or average conditions reported throughout the United States at the time of its preparation. The user must appreciate the possibility that individual locations or areas may differ from the average, and avail himself of additional information for specific problems. The manual does not, therefore, provide rigid standards for capacity measurements, but instead provides a guide in lieu of more detailed information.

The principal characteristics of traffic operation relating to capacity are discussed, for the various highway elements. Field studies and research results are correlated with present highway design practice and rational procedures are developed for analyzing the capacity of existing or proposed facilities. The bibliographies at the end of certain chapters list additional selected references pertaining to highway capacity studies that may be of benefit to the reader.

Highway capacity has been the subject of continuing study over a long period of time, but by no means is the research completed. This summary of present information points out the need for extending the quantity of data and breadth of analysis beyond existing knowledge. Much has been done, but much more study is necessary to accurately define and measure the factors involved in determining the capacity of highways.

One objective of the Highway Capacity Committee in preparing this manual is to encourage continued research in the field of highway capacity. Practitioner, student, and researcher alike, benefit from and contribute to this reservoir of knowledge. The Highway Research Board, through its committees and individual members, extends its assistance, encouragement and advice to persons interested in furthering research in highway capacity.

Rural freeway in rolling terrain. Shoulders provided on both sides of travel lanes. Aesthetic features provided by natural plantings and rock outcrops. Edge stripping provides delineation at edge of travel lane.

CHAPTER TWO

DEFINITIONS

INTRODUCTION

The confusion that has existed concerning the meaning and shades of meaning of many terms used in traffic engineering practice has contributed, in some measure at least, to the wide differences of opinion regarding the capacity of various highway facilities. In fact, the term which is perhaps most widely misunderstood and improperly used in the field of highway capacity is the word "capacity" itself.

The definitions given here are intended to be those most descriptive and most widely used in traffic engineering practice. Most of them are based on current usage or are definitions already adopted by various organizations.

There are, however, cases in which a definition represents a combination of, or compromise between, definitions appearing in previously published material. The Committee's primary attempt has been to ascribe definite meanings to terms as they have been used in this report, thus minimizing likelihood of misinterpretation of its content.

In general, only terms which are used in

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this manual are included. As an aid in locating the definition of any term listed, an alphabetical index of the terms is included at the end of the chapter.

CAPACITY DEFINED

Capacity is the maximum number of vehicles which has a reasonable expectation of passing over a given section of a lane or a roadway in one direction (or in both directions for a two-lane or a three-lane highway) during a given time period under prevailing roadway and traffic conditions. The term *capacity* as used in this manual is synonymous with the term "possible capacity" as used in certain other publications. In the absence of a time modifier, capacity is an hourly volume. The capacity would not normally be exceeded without changing one or more of the conditions that prevail. In expressing capacity, it is essential to state the *prevailing roadway and traffic conditions* under which the capacity is applicable.

The number of vehicles passing a point on a roadway during periods of heavy demand will be governed by one of the following limiting measures:

1. The demand being placed upon the roadway by the vehicles desiring to use it at the particular time.

2. The capacity of the roadway at:
   (a) The point of observation;
   (b) A point upstream; or
   (c) A point downstream.

Urban freeway adjacent to central business district. Grade separations carry city streets over this high-volume full-control-of-access facility.

When item 1 is the limiting measure, the flow is less than the capacity of the observed section or of relatively nearby sections upstream or downstream. When the flow is limited by item 2a, traffic will generally be flowing freely at the point of observation, but a backlog may occur on the section immediately upstream. When the flow is limited by item 2b, traffic will generally be flowing freely at the point of observation because it has been metered at the point upstream. In this case, unless the upstream bottleneck is visible from, or reported to, the point of observation, it is not possible to determine whether condition 1 or condition 2b is the limiting measure. When the flow is limited by item 2c, a backlog will occur on the section under observation. The performance of the particular group of drivers and vehicles at a specific time can influence the flow under conditions 2a, 2b, and 2c.

PREVAILING CONDITIONS

The capacity of a roadway depends on a number of conditions. Composition of traffic, roadway alinement, and number and width of lanes are a few of those conditions which may be referred to collectively as the *prevailing conditions*.

The prevailing conditions may be divided into two general groups—(1) those that are established by the physical features of the roadway, and (2) those that are dependent on the nature of traffic on the roadway.

Grade separation over 8-lane freeway. Paved median provided with edge striping for visibility and fence in center. Note provision of shoulders and adequate side clearances at structure.

Those in the first group, none of which change unless some construction or reconstruction is performed, are referred to as the *prevailing roadway conditions*. Those in the second group, any of which may change or be changed from hour to hour or during various periods of the day, are referred to as the *prevailing traffic conditions*.

In addition to these prevailing roadway and traffic conditions, *ambient conditions* are present during all traffic flows. These conditions relate primarily to weather and include measures such as clear, dry, cold, warm, hot, rain, snow, fog, smog, smoke, wet or icy pavement, and windy. Visibility during different hours of the day, particularly in daylight as compared to dark, also is an ambient condition. These conditions affect the ability of a roadway to accommodate traffic. However, because only very limited data are available to quantify their effect on capacity, they cannot be discussed in detail in this manual.

LEVEL OF SERVICE

Level of service is a term which, broadly interpreted, denotes any one of an infinite number of differing combinations of operating conditions that may occur on a given lane or roadway when it is accommodating various traffic volumes. Level of service is a qualitative measure of the effect of a number of factors, which include speed and travel time, traffic interruptions, freedom to maneuver, safety, driving comfort and convenience, and operating costs. In practice, selected specific levels are defined in terms of particular limiting values of certain of these factors.

A given route or route segment will normally consist of a number of roadway components. In addition to the through lanes, these components may include weaving areas, ramps, ramp terminals, auxiliary lanes and intersections. These various roadway components should provide operating characteristics in harmony with the specified level of service for the through lanes which comprise the basic framework for the overall route or route segment.

A given lane or roadway may provide a wide range of levels of service. The various levels for any specific roadway are functions

Urban freeway with full control of access: diamond interchange in foreground, cloverleaf interchange top center.

of the volume and composition of traffic and of the speeds attained. A lane or roadway designed for a certain level of service at a specified volume will actually operate at many different levels of service as the flow varies during an hour, and as the volume varies during different hours of the day, days of the week, periods of the year, and during different years with traffic growth. Further, different types of highways, roads and streets, such as freeways, expressways at grade, major multilane highways, local two-lane rural roads, urban arterial streets, and downtown streets, nearly always provide different levels of service that cannot be directly related to one another because each must be measured by a different standard or scale.

From the viewpoint of the driver, low

flow rates or volumes on a given lane or roadway provide higher levels of service than greater flow rates or volumes on the same lane or roadway. Thus, the level of service for any particular lane or roadway varies inversely as some function of the flow or volume, or of the density.

This manual includes narrative descriptions of prevailing traffic flow conditions which represent several levels of service. These levels encompass a working range of volumes from a condition of free flow to a condition of capacity. It is the intent of the Committee that this manual provide guidelines from which the user can select a volume which corresponds to the level of service best adapted to the specific need.

SERVICE VOLUME

A *service volume* is the maximum number of vehicles that can pass over a given section of a lane or roadway in one direction on multilane highways (or in both directions on a two- or three-lane highway) during a specified time period while operating conditions are maintained corresponding to the selected or specified level of service. In the absence of a time modifier, service volume is an hourly volume.

OTHER DEFINITIONS

Other terms which are used in this manual and their definitions follow, not in alphabetical order, but grouped according to the subject to which they are most closely related. For convenience in locating any definition an alphabetical index is included at the end of the chapter.

ROADWAY DEFINITIONS

1. General

a. Highway, street, or *road.*—These are general terms denoting a public way for purposes of vehicular and pedestrian travel, including the entire area within the right-of-way. In rural areas, or in urban areas where there is comparatively little access and egress, a way between prominent termini is usually called a *highway* or a *road*. A way in an urban area, with or without provision made for curbs, sidewalks, and paved gutters, is ordinarily called a *street*.

b. Control of access.—The condition where the right of owners or occupants of abutting land or other persons to access, light, air or view in connection with a highway is fully or partially controlled by public authority.

(1) *Full control of access* means that the authority to control access is exercised to give preference to through traffic by providing access connections with selected public roads only and by prohibiting crossings at grade or direct private driveway connections.

(2) *Partial control of access* means that the authority to control access is exercised to give preference to through traffic to a degree that, in addition to access connections with selected public roads, there may be some crossings at grade and some private driveway connections.

(3) *Uncontrolled access* means that the authority having jurisdiction over a highway, street, or road, does not limit the number of points of ingress or egress except through the exercise of control over the placement and the geometrics of connections as necessary for the safety of the traveling public.

2. Functional Types

a. Arterial highway.—A highway primarily for through traffic, usually on a continuous route.

b. Expressway.—A divided arterial highway for through traffic with full or partial control of access and generally with grade separations at major intersections.

c. Freeway.—An expressway with full control of access.

d. Parkway.—An arterial highway for noncommercial traffic, with full or partial control of access, and usually located within a park or a ribbon of parklike developments.

e. Major street or *major highway.*—An arterial highway with intersections at grade and direct access to abutting property, and on which geometric design and traffic control measures are used to expedite the safe movement of through traffic.

f. Through street or *through highway.*—Every highway or portion thereof at the

Fourteen traffic lanes are provided on this depressed urban freeway. Frequent grade separations provide for cross traffic movement. Note diamond interchange ramps intersecting with one-way frontage roads, and lane separators.

entrance to which vehicular traffic from intersecting highways is required by law to stop or yield before entering or crossing and where appropriate signs are erected as provided by law, unless entry or crossing is made on the proper indication of a traffic control signal.

g. Local street or *local road.*—A street or road primarily for access to residence, business, or other abutting property.

3. Directional Use

a. One-way road.—A road on which the movement of traffic is confined to one direction.

b. Two-way road.—A road on which traffic may move in opposing directions si-

multaneously. It may be either divided or undivided.

4. Cross-Section Components

a. Roadway.—That portion of a road which is improved, designed, or ordinarily intended for vehicular use. Divided roads, and roads with frontage roads, have more than one roadway. On undivided roads without frontage roadways, the roadway width normally lies between the regularly established curb lines or between the outer extremities of the shoulders, whichever is appropriate.

b. Frontage road.—A road contiguous to and generally paralleling an expressway, freeway, parkway, or through street and so

designed as to intercept, collect, and distribute traffic desiring to cross, enter, or leave such highway and which may furnish access to property that otherwise would be isolated as a result of the controlled-access feature; sometimes called a *service road*.

c. Pavement.—That part of a roadway having a constructed surface for the facilitation of vehicular movement.

d. Shoulder.—That portion of a roadway between the outer edge of the through traffic pavement and the curb or the point of intersection of the slope lines at the outer edge of the roadway and the fill, ditch, or median slope, for the accommodation of stopped vehicles, for emergency use, and for lateral support.

e. Curb.—A vertical or sloping member along the edge of a pavement or shoulder forming part of a gutter, strengthening or protecting the edge, and clearly defining the edge to vehicle operators. The surface of the curb facing the general direction of the pavement is called the "face."

f. Separator.—An area or a device located longitudinally between two roadways so as to separate traffic flowing in the same or opposite directions and so designed as to discourage or prevent passage by vehicles from the traffic lanes on one side of the separator to those on the other.

(*1*) *Median.*—That portion of a divided highway separating the traveled ways for traffic in opposite directions.

(*2*) *Lane separator.*—A separator between traffic streams moving in the same direction where the service rendered by the roadways on either side of the separator is essentially of the same character, as distinguished from that of a frontage road.

(*3*) *Outer separator.*—A separator between a frontage road and the roadway of a controlled-access highway or major street.

(*4*) *Traffic island.*—An island provided in the roadway to separate or direct streams of traffic; includes both divisional and channelizing islands.

g. Auxiliary lane.—The portion of roadway adjoining the traveled way for parking, speed change, or for other purposes supplementary to the through traffic movement.

(*1*) *Acceleration lane.*—A speed change lane for the purpose of:

(a) Enabling a vehicle entering a roadway to increase its speed to a rate at which it can more safely merge with through traffic.

(b) Providing the necessary merging distance.

(c) Giving the main roadway traffic the necessary time and distance to make appropriate adjustments.

(*2*) *Deceleration lane.*—A speed change lane for the purpose of enabling a vehicle that is to make an exit turn from a roadway to slow to the safe speed on the curve ahead after it has left the main stream of faster-moving traffic.

(*3*) *Parking lane.*—An auxiliary lane primarily for the parking of vehicles.

(*4*) *Climbing lane.*—An auxiliary lane in the upgrade direction intended for use primarily by slow-moving vehicles to maintain capacity and freedom of operation.

5. Cross-Sectional Design

a. Undivided road.—A road which has no directional separator, either natural or structural, separating traffic moving in opposite directions.

b. Divided road.—A two-way road on which traffic in one direction of travel is separated from that in the opposite direction. Such a road has two or more roadways.

6. Width, in Lanes

a. Two-lane road.—An undivided two-way road having one lane for traffic in each of two opposing directions.

b. Three-lane road.—An undivided two-way road providing one lane for the exclusive use of traffic in each of two opposing directions and a third (center) lane usually for use by traffic in either direction in overtaking and passing. In special cases, the center lane may be operated reversibly, or reserved for left turns only.

c. Multilane road.—A road having two or more lanes for traffic in each direction, or four or more lanes for traffic in two directions. It may be one-way or two-way, divided or undivided.

7. *Traffic Lane.*—A strip of roadway intended to accommodate a single line of moving vehicles.

a. *Right lane* or *lane one.*—On any roadway, the lane on the extreme right, in the direction of traffic flow, available for moving traffic. Sometimes referred to as the *outside lane* on rural highways or the *curb lane* on city streets.

b. *Left lane.*—On a two-lane, two-way road, that traffic lane which is to the left of the center line and which is normally used by traffic in the opposite direction; or on a multilane road, the extreme left traffic lane of those available for traffic traveling in one direction.

c. *Center lane.*—On an undivided two-way road having an odd number of traffic lanes, the lane which may be used by traffic traveling in either direction, or which may be operated reversibly or reserved for left turns.

d. *Lane two, lane three,* etc.—On a multilane roadway, the traffic lane or lanes to the left of the right lane (or lane one) available for traffic traveling in the same direction, designated "lane two," "lane three," etc., in that order numbered from right to left when facing in the direction of traffic flow.

e. *Reversible lane* or *lanes.*—A lane or lanes where traffic moves in one direction only during some period of time, then in the reverse direction during another period of time.

f. *Left-turn lane.*—A traffic lane within the normal surfaced width of a roadway, or an auxiliary lane adjacent to or within a median, reserved for left-turning vehicles at an intersection.

g. *Right-turn lane.*—A traffic lane within the normal surfaced width of a roadway, or an auxiliary lane to the right of and adjacent to the through traffic lanes, reserved

Expressway at grade, with partial control of access. Note median openings and lane provided for left-turn vehicles. Grade separation (in background) carries cross traffic.

for right-turning vehicles at an intersection.

h. Bus lane.—A traffic lane reserved for buses, either in transporting, discharging, or loading passengers, except, in the case of a curb bus lane, for use by turning vehicles at intersection approaches.

8. Intersection.—The area embraced within the prolongation or connection of the lateral curb lines, or, if none, then the lateral boundary lines of the roadways of two highways which join one another at, or approximately at, right angles, or the area within which vehicles traveling on different highways joining at any other angle may come in conflict.

a. Intersection leg.—That part of any one of the roadways radiating from an intersection which is outside the area of the intersection proper.

> *(1) Approach.*—That portion of a leg which is used by traffic approaching the intersection.
> *(2) Exit.*—That portion of a leg which is used by traffic in leaving an intersection.

b. Three-leg intersection.—A roadway intersection with three intersection legs. If one of these intersection legs is an approximate prolongation of the direction of approach of another, and if the third leg intersects this prolongation at an angle between 75° and 105°, the three-way intersection is classed as a T intersection. If one leg is an approximate prolongation of the approach of another, and the third leg intersects this prolongation at an angle less than 75° or greater than 105°, it is classed as a Y intersection.

c. Four-leg intersection.—A roadway intersection with four intersection legs. If two of the intersection legs are approximate prolongations of the other two legs, and the angle of intersection of these prolongations is 75° or more, but not greater than 105°, it is classed as a four-way right-angled intersection. If two of the intersection legs are approximate prolongations of the directions of approach of the other two, and the angle of intersection of these two prolongations is less than 75° or more than 105°, it is classed as a four-way oblique intersection.

d. Multi-leg intersection.—An intersection having five or more legs.

e. Rotary intersection.—A confluence of three or more intersection legs at which traffic merges into and emerges from a one-way roadway in a counterclockwise direction around a central area. (In those few countries where "keep-to-the-left" driving rules apply, traffic moves clockwise.)

9. Channelization.—The separation or regulation of conflicting traffic movements into definite paths of travel by use of pavement markings, raised islands, or other suitable means to facilitate the safe and orderly movement of traffic, both vehicular and pedestrian.

10. Highway Grade Separation.—A structure used to separate vertically two or more intersecting roadways, thus permitting traffic on all roads to cross traffic on all other roads without interference.

a. Interchange.—A system of interconnecting roadways in conjunction with one or more grade separations, providing for the interchange of traffic between two or more roadways or highways on different levels.

b. Ramp.—An interconnecting roadway of a traffic interchange, or any connection between highways at different levels, or between parallel highways, on which vehicles may enter or leave a designated roadway.

> *(1) Inner loop.*—A ramp used by traffic destined for a left-turn movement from one of the through roadways to a second when such movement is accomplished by making a right-exit turn followed by a three-quarter-round right-turn maneuver and a right-entrance turn.
> *(2) Outer connection.*—A ramp used by traffic destined for a right-turn movement from one of the through roadways separated by a structure to the second through roadway.
> *(3) Direct connection.*—A form of ramp which does not deviate appreciably from the intended direction of travel. The inner loop for left-turning movement is avoided by the use of separate structures. An outer connec-

Rotary intersection in urban area. Grade separation used to carry major traffic flow underneath. Traffic signals, pavement markings, and raised islands extensively employed to facilitate traffic movement.

tion is a direct connection for right-turning movements.

c. Ramp terminal.—The general area where a ramp connects with a roadway. Ramps have both entrance and exit terminals. The entrance terminal relates to a merging condition; the exit terminal relates to a diverging condition.

11. Weaving Section.—A length of one-way roadway at one end of which two one-way roadways merge and at the other end of which they separate. A multiple weaving section involves more than two entrance and/or exit roadways.

12. Sight Distance.—The distance visible to the driver of a passenger vehicle, meas-

ured along the normal travel path of a roadway, to the roadway surface or to a specified height above the roadway, when the view is unobstructed by traffic.

a. Stopping sight distance.—The distance required by a driver of a vehicle, traveling at a given speed, to bring his vehicle to a stop after an object on the roadway becomes visible. It includes the distance traveled during the perception and reaction times and the vehicle braking distance.

b. Passing sight distance.—The minimum sight distance on two- and three-lane highways that must be available to enable the driver of one vehicle to pass another vehicle safely and comfortably

without interfering with the speed of an on-coming vehicle traveling at the design speed, should it come into view after the overtaking maneuver is started.

13. Terrain.—The topography of the profile of a highway, road, or street. As used in this manual, the term generally has one of three modifiers: level, rolling, or mountainous. These three modifiers represent combinations of geometric features in varying degrees which relate primarily to gradients and horizontal and vertical alinement. They reflect the effect on capacity of the operating characteristics of trucks in terms of their passenger car equivalent under the different geometric conditions.

a. *Level terrain.*—Any combination of gradients, length of grade, or horizontal or vertical alinement that permits trucks to maintain speeds that equal or approach the speeds of passenger cars.

b. *Rolling terrain.*—Any combination of gradients, length of grade, or horizontal or vertical alinement that causes trucks to reduce their speeds substantially below that of passenger cars on some sections of the highway, but which does not involve sustained crawl speed by trucks for any substantial distance.

c. *Mountainous terrain.*—Any combination of gradients, length of grade, or horizontal or vertical alinement that will cause trucks to operate at crawl speed for considerable distances or at frequent intervals.

d. *Sustained grade.*—A continuous highway grade of appreciable length and consistent or nearly consistent gradient.

14. Ideal Conditions.—The base conditions as used in capacity determinations, including:

a. Uninterrupted flow, free from side interferences of vehicles and pedestrians.

b. Only passenger cars in the traffic stream.

c. Traffic lanes 12 ft wide with adequate shoulders and no obstructions within 6 ft of the edge of the pavement.

d. Horizontal and vertical alinement satisfactory for average highway speeds of 70 mph and no restricted passing sight distance on two- and three-lane highways.

TRAFFIC CONTROL DEVICE DEFINITIONS

1. Traffic Control Device.—Any sign, signal, marking, or device placed or erected for the purpose of regulating, warning, or guiding vehicular traffic and/or pedestrians.

2. Pavement Markings

a. *Lane line.*—A line' separating two lanes for traffic moving in same direction.

b. *Center line.*—A line indicating the division of the pavement between traffic moving in opposite directions. It is not necessarily at the exact geometric center of the pavement.

3. Traffic Sign.—A traffic control device mounted on a fixed or portable support which conveys a specific message by means of words or symbols, and is officially erected for the purpose of regulating, warning, or guiding traffic.

4. Traffic Control Signal.—Any device, whether manually, electrically, or mechanically operated, by which traffic is alternately directed to stop and permitted to proceed.

a. *Signal indication.*—The illumination of a traffic signal lens or equivalent device or a combination of several lenses or equivalent devices at the same time.

b. *Time cycle.*—The time period required for one complete sequence of signal indications.

c. *Interval.*—Any one of the several divisions of the time cycle during which signal indications do not change.

d. *Phase.*—A part of the time cycle allocated to any traffic movement or to any combination of traffic movements receiving the right-of-way simultaneously during one or more intervals.

e. *Pretimed signal.*—A type of traffic control signal which directs traffic to stop and permits it to proceed in accordance with predetermined time schedules.

f. *Traffic-actuated signal.*—A type of traffic control signal in which the intervals are varied in accordance with the demands of traffic as registered by the actuation of detectors.

(1) *Semi-traffic-actuated signal.* — A type of traffic-actuated signal in which means are provided for traffic actuation

on one or more, but not all, approaches to the intersection.

(2) *Full traffic-actuated signal.*—A type of traffic-actuated signal in which means are provided for traffic actuation on all approaches to the intersection.

(3) *Pedestrian-actuated signal.* — A type of traffic control signal which may be actuated by a pedestrian.

g. *Progressive system.*—A signal system in which the successive signal faces controlling a given street give "go" indications in accordance with a time schedule to permit (as nearly as possible) continuous operation of groups of vehicles along the street at a planned rate of speed, which may vary in different parts of the system.

TRAFFIC DEFINITIONS

1. Traffic.—All types of conveyances, together with their load, either singly or as a whole, as well as pedestrians, while using any roadway for the purpose of transportation or travel.

a. *Vehicle.*—Any component of wheeled traffic. Unless otherwise qualified, the term vehicle will normally apply to free-wheeled vehicles as hereinafter defined.

b. *Free-wheeled vehicle.*—Any component of traffic not limited in its field of operation to rails or tracks.

c. *Passenger car.*—A free-wheeled, self-propelled vehicle generally designed for the transportation of persons, but limited in seating capacity to not more than nine passengers, including taxicabs, limousines, and station wagons. Also included, for capacity purposes, are two-axle, four-tired pickups, panels and light trucks, which have operating characteristics similar to those of passenger cars, but not motorcycles.

d. *Truck.*—A free-wheeled vehicle having dual tires on one or more axles, or having more than two axles, designed for the transportation of cargo rather than passengers. Includes tractor-trucks, trailers and semitrailers when used in combination. Excludes those two-axle, four-tired vehicles that may be classified as a truck for registration purposes, but which have operating characteristics similar to those of a passenger car.

e. *Bus.*—A free-wheeled vehicle having a self-contained source of motive power, designed for the transportation of persons, and having a seating capacity of ten or more passengers.

f. *Commercial vehicle.*—A truck or a bus.

TRAFFIC OPERATIONS DEFINITIONS

1. Speed.—The rate of movement of vehicular traffic or of specified components of traffic, expressed in miles per hour.

a. *Spot speed.*—The speed of a vehicle as it passes a specified point on a roadway.

b. *Average spot speed.*—The average of the individual spot speeds of all vehicles or a specified class of vehicles at a specific point on a given roadway during a specified period of time. Also referred to as *time mean speed.*

c. *Overall travel speed.*—The total distance traversed divided by the total time required, including all traffic delays.

d. *Average overall travel speed.*—The summation of distances traveled by all vehicles or a specified class of vehicles over a given section of highway during a specified period of time, divided by the summation of overall travel times.

e. *Space mean speed.*—The average of the speeds of vehicles within a given space or section of roadway at a given instant. Also, the average speed of a specified group of vehicles based on their average travel time over a section of roadway.

f. *Design speed.*—A speed selected for purposes of design and correlation of those features of a highway, such as curvature, superelevation, and sight distance, upon which the safe operation of vehicles is dependent.

g. *Average highway speed.* — The weighted average of the design speeds within a highway section, when each subsection within the section is considered to have an individual design speed.

h. *Operating speed.*—The highest overall speed at which a driver can travel on a given highway under favorable weather conditions and under prevailing traffic conditions without at any time exceeding the safe

speed as determined by the design speed on a section-by-section basis.

i. Free-flow operating speed.—The operating speed of a passenger car over a section of highway during extremely low traffic densities.

j. Running speed.—The speed over a specified section of highway, being the distance divided by running time. The average for all traffic, or a component thereof, is the summation of distances divided by the summation of running times.

2. *Running Time.*—The time the vehicle is in motion.

3. *Delay.*—The time consumed while traffic or a specified component of traffic is impeded in its movement by some element over which it has no control. Usually expressed in seconds per vehicle.

a. Fixed delay.—The delay to which vehicles are subjected during light traffic volumes or low densities. The delays experienced by a lone vehicle as a result of traffic signals or stop signs are fixed delays.

b. Operational delay.—The delay caused by the interference between components of traffic. The difference between travel times over a route during extremely low and high traffic volumes is operational delay. The time consumed while waiting at a stop sign for cross traffic to clear, the time losses resulting from congestion, from interference with parking vehicles, and from turning vehicles are examples of operational delays.

4. *Vehicular Gap.*—The interval in time or distance between individual vehicles measured from the rear of one vehicle to the head of the following vehicle.

5. *Spacing.*—The interval in distance from head to head of successive vehicles.

6. *Headway.*—The interval in time between individual vehicles measured from head to head as they pass a given point.

7. *Weaving.*—The crossing of traffic streams moving in the same general direction accomplished by merging and diverging.

8. *Merging.*—The process by which two separate traffic streams moving in the same general direction combine or unite to form a single stream. The total volume in this combined stream is the *merge volume.*

9. *Diverging.*—The dividing of a single stream of traffic into separate streams. The total volume in this single stream before division is the *diverge volume.*

10. *Volume.*—The number of vehicles that pass over a given section of a lane or a roadway during a time period of one hour or more. Volume can be expressed in terms of daily traffic or annual traffic, as well as on an hourly basis.

a. Average annual daily traffic.—The total yearly volume divided by the number of days in the year, commonly abbreviated as AADT.

b. Average daily traffic.—The total volume during a given time period in whole days greater than one day and less than one year divided by the number of days in that time period, commonly abbreviated as ADT.

c. Maximum annual hourly volume.—The highest hourly volume that occurs on a given roadway in a designated year.

d. Tenth, twentieth, thirtieth, etc., highest annual hourly volume.—The hourly volume on a given roadway that is exceeded by 9, 19, 29, etc., respectively, hourly volumes during a designated year.

11. *Peak-Hour Traffic.*—The highest number of vehicles found to be passing over a section of a lane or a roadway during 60 consecutive minutes.

12. *Rate of Flow.*—The hourly representation of the number of vehicles that pass over a given section of a lane or a roadway for some period less than one hour. It is obtained by expanding the number of vehicles to an hourly rate by multiplying the number of vehicles during a specified time period by the ratio of 60 min to the number of minutes during which the flow occurred. The term "rate of flow" will normally be prefixed by the time period for the measurement. For example, a 15-min count of N vehicles multiplied by 60/15 or 4 would produce a "15-min rate of flow of $4N$ vehicles per hour."

13. *Interrupted Flow.*—A condition in which a vehicle traversing a section of a lane

High-density traffic on parkway. Relatively unusual condition of complete congestion in both directions due to bottleneck conditions (not shown).

or a roadway is required to stop by a cause outside the traffic stream, such as signs or signals at an intersection or a junction. Stoppage of vehicles by causes internal to the traffic stream does not constitute interrupted flow.

14. Uninterrupted flow.—A condition in which a vehicle traversing a section of a lane or a roadway is not required to stop by any cause external to the traffic stream although vehicles may be stopped by causes internal to the traffic stream.

15. Density.—The number of vehicles occupying a unit length of the through traffic lanes of a roadway at any given instant. Usually expressed in vehicles per mile.

    a. Average density.—The average number of vehicles per unit length of roadway over a specified period of time.

    b. Critical density.—The density of traffic when the volume is at capacity on a given roadway. At a density either greater or less than the critical density, the volume of traffic will be decreased. Critical density occurs when all vehicles are moving at about the same speed.

16. Load Factor.—A ratio of the total number of green signal intervals that are fully utilized by traffic during the peak hour to the total number of green intervals for that approach during the same period. Its maximum attainable value is one.

17. Peak-Hour Factor.—A ratio of the volume occurring during the peak hour to the maximum rate of flow during a given time period within the peak hour. It is a measure of peaking characteristics, whose maximum attainable value is one. The term must be qualified by a specified short period within the hour; this is usually 5 or 6 min for free-
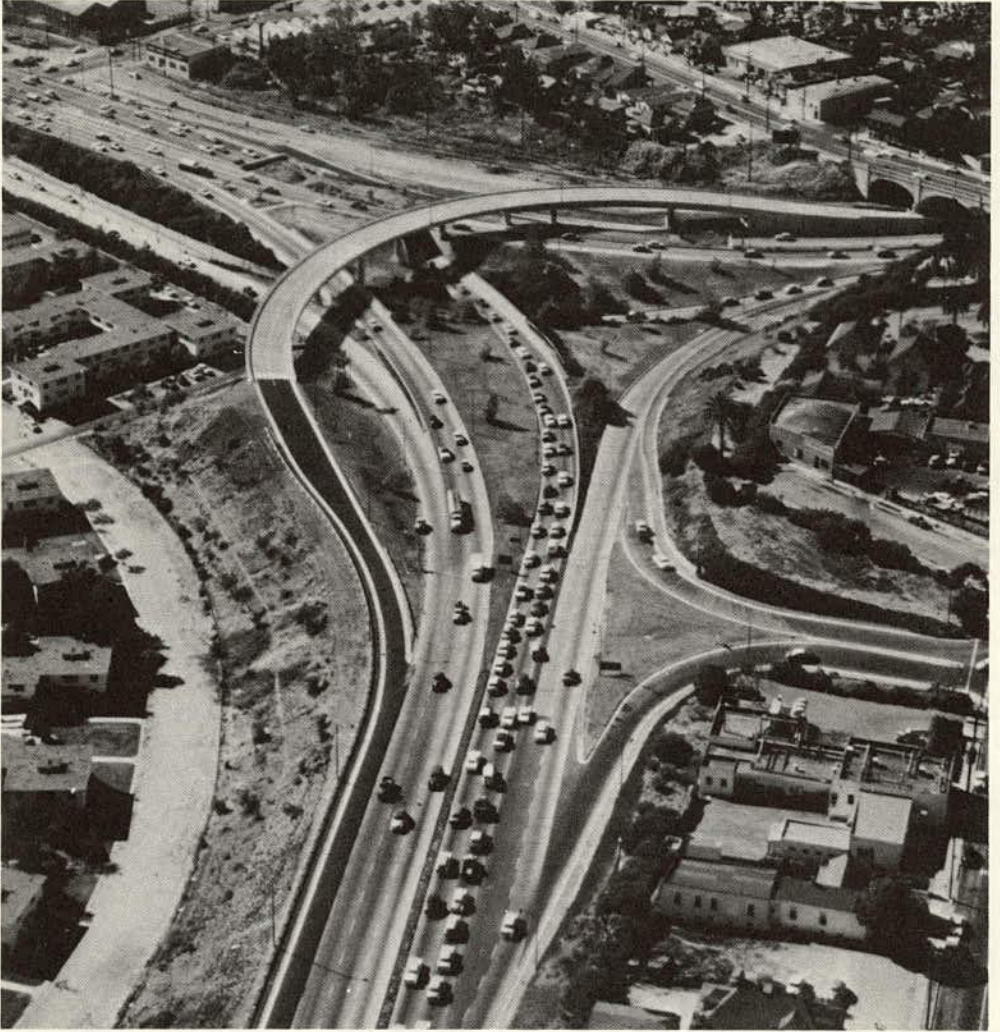
Unbalanced traffic flow. Predominant flow is from bottom to top of photo; counter movement is relatively light. Note direct left-turn connection from highway at right and channelization used to separate ramps at right center.

way operation and 15 min for intersection operation. For example, "a peak-hour factor of 0.80 based on a 5-min rate of flow."

18. Friction

a. Intersectional friction.—The retarding effect on traffic movement caused by potential and actual traffic movement conflicts at an intersection or merge of two moving streams of traffic. This friction is due solely to the effect of one stream of traffic crossing the other stream.

b. Marginal friction.—The retarding effect on the free flow of traffic caused by interference of any sort at the margin of the highway. This does not include conflicts at intersections or medial friction.

c. Medial friction.—The retarding

effect on the free flow of traffic caused by interference between traffic units proceeding in opposite directions on a highway. (Turning conflicts are classed as intersectional conflicts).

d. Stream friction.—The retarding effect on the free flow of traffic caused by mutual interferences between traffic units proceeding in the same direction. This does not include turning conflicts. Conflicts are caused primarily by differences in sizes and speeds of traffic units.

19. Upstream.—The direction along the roadway from which the vehicle flow under consideration has come.

20. Downstream.—The direction along the roadway toward which the vehicle flow under consideration is moving.

21. Bottleneck.—A constriction along a traveled way which limits the amount of traffic which can proceed downstream from its location.

22. Passenger Car Equivalent.—The number of passenger cars displaced in the traffic flow by a truck or a bus, under the prevailing roadway and traffic conditions.

23. Platoon.—A closely grouped elemental component of traffic, composed of several vehicles, moving or standing ready to move over a roadway, with clear spaces ahead and behind.

24. Base Volume.—A volume value required for certain computational purposes, which differs from capacity under prevailing conditions only in that the adjustment factors applied to capacity under ideal conditions to derive it are those for a particular level of service rather than those for capacity.

LAND USE AND DEVELOPMENT DEFINITIONS

1. Central Business District.—That portion of a municipality in which the dominant land use is for intense business activity. This district is characterized by large numbers of pedestrians, commercial vehicle loadings of goods and people, a heavy demand for parking space, and high parking turnover.

Narrow 10-ft lanes, lack of shoulders, restrictive side clearances, and lack of proper sight distance reduce capacity substantially on this two-lane rural highway.

2. Fringe Area.—That portion of a municipality immediately outside the central business district in which there is a wide range in type of business activity, generally including small businesses, light industry, warehousing, automobile service activities, and intermediate strip development, as well as some concentrated residential areas. Most of the traffic in this area involves trips that do not have an origin or a destination within the area. This area is characterized by moderate pedestrian traffic and a lower parking turnover than is found in the central business district, but it may include large parking areas serving that district.

3. Outlying Business District.—That portion of a municipality or an area within the influence of a municipality, normally separated geographically by some distance from the central business district and its fringe area, in which the principal land use is for business activity. This district has its own local traffic circulation superimposed on through movements to and from the central business district, a relatively high parking demand and turnover, and moderate pedestrian traffic. Compact off-street shopping developments entirely on one side of the street are not included in the scope of this definition.

4. Residential Area.—That portion of a municipality, or an area within the influence of a municipality, in which the dominant land use is residential development, but where small business areas may be included. This area is characterized by few pedestrians and a low parking turnover.

INDEX OF DEFINITIONS

A signalized intersection in a central business district.

TRAFFIC CHARACTERISTICS

INTRODUCTION

The capacity of a highway * is a measure of its ability to accommodate traffic. Obviously, this ability depends greatly on the physical features of the roadway itself. Yet there are other factors not directly related to roadway features that are of major importance in determining the capacity of any highway. Many of these factors relate to variations in the traffic demand and the interaction of vehicles in the traffic stream. Thus, highway capacity is a function of the physical features of the highway and the operational characteristics of the traffic thereon.

In general terms, the aggregate demand for the use of a highway is expressed in traffic volume, whereas the level of traffic service to the road user is a function of comfort and convenience, speed, travel time, maneuverability, safety, and cost. The broad field of highway capacity involves determination of whether or not a certain roadway is capable of handling the predicted or measured demand at an acceptable level of service.

This chapter first discusses volume, which is a manifestation of traffic demand. The demand for the use of any highway is a changing quantity and exhibits many types of variations. Although a highway may have only one capacity (unless prevailing conditions change), in practice it will serve a wide range of traffic volumes under varying operating conditions. Thus, capacity considerations should be based not only on whether a highway can handle a selected volume at an acceptable level of service, but also on whether the range in volumes that occurs can be handled within an accept-

able *range* in the resulting levels of service. Variations in demand do not, in themselves, represent variations in capacity; rather, they result in variations in the level of service which a road provides.

The chapter then discusses speed trends and variations, inasmuch as speed and travel time are significant qualitative measures of levels of traffic service.

The remaining portion of the chapter deals with the interrelationship of speed, volume, and vehicular spacings in connection with their effect on roadway capacity.

MAXIMUM OBSERVED TRAFFIC VOLUMES

Tables 3.1 through 3.10 give maximum observed traffic volumes recorded in 1961 on various highways throughout the United States, as reported by state, city, and other highway officials. Highways have been classified by width, type, and location within an urban or rural area. For each classification the five highways with the highest hourly volumes were selected for inclusion in these tables, where data were available. In some cases the selection was from a much larger number of locations; for other classes five were not available and only a smaller number could be included. There may well be other highways in some of the classifications where volumes exceed those shown but for which no data have been reported.

The traffic volume for each specific location reported is the highest for which accurate data are available. In some cases the reported hourly volumes probably represent capacity, whereas for other locations the hourly volumes were merely the highest that had been observed. On some of these highways the traffic demand has probably been sufficient to reach or exceed the same volumes on many occasions. For others, the maximum value which was recorded on

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* The term "highway" is used in this chapter in its general sense, denoting any public way for purposes of vehicular travel.

TABLE 3.1—HIGHEST REPORTED HOURLY VOLUMES ON TWO-LANE, TWO-WAY HIGHWAYS IN THE UNITED STATES, 1961

ROUTE AND LOCATION	AVERAGE LANE WIDTH (FT)	VOLUME (VPH)		ADT FOR BOTH DIRECTIONS
		HEAVY DIRECTION	BOTH DIRECTIONS	
(a) URBAN EXPRESSWAYS AT GRADE				
N.J. 208, Fairlawn, Bergen Co., N.J.	12.0	1,090	2,056	16,028
P.R. 21, San Juan, Puerto Rico	9.1	—	1,482	19,201
(b) RURAL HIGHWAYS				
Md. 5, Woods Corner, Prince Georges Co., Md.	12.0	1,099	1,871	18,825
Md. 26 (Liberty Road), Baltimore, Md.	10.0	1,224	1,777	21,500
U.S. 40, West of Denver, Colo.	12.0	—	1,760	5,950
Md. 3, Glen Burnie, Anne Arundel Co., Md.	12.0	855	1,680	22,275
Del. 141, New Bridge Rd., New Castle Co., Del.	12.0	963	1,605	15,935
(c) MAJOR CITY STREETS				
U.S. 95, Bonanza Road, Las Vegas, Nev.	12.0	—	2,297	20,064
U.S. 60, Washington St., Charleston, W.Va.	15.0	1,125	2,062	19,850
U.S. 27, Clinton St., Ft. Wayne, Ind.	10.0	1,063	2,024	20,041
Coldwater Canyon Dr., Los Angeles, Calif.	15.0	1,586	1,985	15,000
Rt. TT, Brown Road, St. Louis Co., Mo.	11.0	1,223	1,970	—
(d) BRIDGES AND TUNNELS				
Posey Tube, Oakland, Calif.	11.0	1,303	2,595	27,163
Lake St. Bridge, Minneapolis-St. Paul, Minn.	14.0	1,515	2,570	25,024
Broadway Ave. Bridge, Minneapolis-St. Paul, Minn.	14.0	1,498	2,373	17,956
C & O Bridge, Cincinnati, Ohio	13.0	1,397	2,281	31,088
Plymouth Ave. Bridge, Minneapolis-St. Paul, Minn.	15.0	1,182	2,262	14,062

TABLE 3.2—HIGHEST REPORTED HOURLY VOLUMES ON THREE-LANE, TWO-WAY HIGHWAYS IN THE UNITED STATES, 1961

ROUTE AND LOCATION	AVERAGE LANE WIDTH (FT)	VOLUME (VPH)		ADT FOR BOTH DIRECTIONS
		HEAVY DIRECTION	BOTH DIRECTIONS	
(a) RURAL HIGHWAYS				
U.S. 16, Fowlerville, Livingston Co., Mich.	10.0	—	1,876	9,900
N.Y. 101, Nassau Co., N.Y.	10.0	—	1,833	18,000
U.S. 302, 1 mi. E. of U.S. 2, Berlin, Vt.	10.0	—	1,434	7,903
Rt. 99, Kamehameha Hwy., Haw.	10.0	697	1,286	10,608
Wis. 38, S. Howell Ave., Milwaukee Co., Wis.	10.0	817	1,260	—
(b) MAJOR CITY STREETS				
Wis. 20, Washington Ave., Racine, Wis.	11.0	1,120	2,205	26,752
Wis. 100, Milwaukee Co., Wis.	10.0	1,014	1,910	—
U.S. 11, N.E. of 4th Ave., Birmingham, Ala.	12.0	1,377	1,812	18,850
U.S. 23, Washtenau Ave., Ann Arbor, Mich.	12.0	1,120	1,723	18,000
Memorial Drive, Atlanta, Ga.[a]	10.6	1,188	1,626	19,500
(c) BRIDGES AND TUNNELS				
Bay Street Viaduct, Savannah, Ga.[a]	10.0	1,690	2,409	22,500
Rt. 37, Dover Twp., Ocean Co., N.J.[b]	9.5	1,813	2,383	9,292
Douglas MacArthur Bridge, E. St. Louis, Ill.	10.0	1,286	1,961	18,800
Los Alamos Canyon Bridge, Los Alamos, N.M.[a]	13.3	—	1,942	8,500
Winooski River Bridge, U.S. 7, Burlington, Vt.	14.0	—	1,862	17,049

[a] Two lanes reserved for heavy flow during peak periods. Volumes given are for unbalanced operation; ADT for all conditions of operation.
[b] Operates two-lane, two-directional except during summer peak periods. Volumes given are for unbalanced operation with two lanes reserved for heavy flow; ADT for all conditions of operation.

TABLE 3.3—HIGHEST REPORTED HOURLY VOLUMES ON FOUR-LANE, TWO-WAY HIGHWAYS [a] IN THE UNITED STATES, 1961

ROUTE AND LOCATION	AVERAGE LANE WIDTH (FT)	AVG. VOLUME (VPH/LANE)		ADT FOR BOTH DIRECTIONS
		LIGHT DIRECTION	HEAVY DIRECTION	
(a) URBAN FREEWAYS				
U.S. 40 (Trk.), Red Feather Expressway, St. Louis, Mo.	12.0	862	2,030	—
No. Sacramento Freeway, Sacramento, Calif.	12.0	860	1,900	64,000
Eastshore Freeway, Oakland, Calif.	12.0	1,315	1,850	66,000
Atlanta Expressway (N.E. Section), Atlanta, Ga.	12.0	950	1,800	50,300
Conn. 15, E. of Silver Lane, Hartford, Conn.	12.0	—	1,794	36,000
(b) RURAL FREEWAYS				
Shirley Highway, Arlington, Va.	12.0	789	1,684	60,400
I-96, Grand River, Livingston Co., Mich.	12.0	214	1,518	15,200
Rt. 128, Circumferential Highway, Newton, Mass.[b]	12.0	1,070	1,435	38,259
New Hampshire Turnpike, Hampton, N.H.	12.0	224	1,144	12,706
I-94, 6 mi. W. of U.S. 24, Wayne Co., Mich.	12.0	258	1,112	24,263
(c) URBAN EXPRESSWAYS AT GRADE				
Lake Shore Drive, S. of 57th Drive, Chicago, Ill.	12.0	445	2,236	75,000
N.J. 4, Paramus, N.J.[c]	10.0	1,438	1,498	62,480
Olentangy River Road, Columbus, Ohio	14.4	851	1,345	42,259
U.S. 6, West 6th Ave., Denver, Colo.	12.0	587	1,177	30,000
U.S. 6, N. of Denver, Colo.	12.0	432	1,107	26,300
(d) RURAL HIGHWAYS				
N.J. 3, Clifton, Passaic Co., N.J.	12.0	—	1,774	40,800
Rt. 90, Pearl City to Aiea, Haw.	11.0	739	1,289	37,728
U.S. 46, Ledgewood, N.J.	12.0	—	1,220	25,932
U.S. 75, W. of Galveston, Tex.	11.0	590	1,193	20,170
(e) MAJOR CITY STREETS				
Sepulveda Blvd., S. of Mulholland Dr., Los Angeles, Calif.	12.5	737	1,742	45,000
U.S. 12, Wayzata Blvd., Minneapolis, Minn.	14.0	420	1,431	32,145
Fla. 9, 27th Ave., N.W., Miami, Fla.	11.5	785	1,195	43,851
Charles St., Baltimore, Md.	9.8	379	1,174	—
Aurora Ave., Seattle, Wash.	11.0	294	1,152	35,758
(f) BRIDGES AND TUNNELS				
U.S. 99, Battery St. Subway, Seattle, Wash.	12.5	314	2,189	49,500
American River Bridge, Sacramento, Calif.[c]	11.0	695	1,850	64,000
Caldecott Tunnel Approach, Oakland, Calif.[c]	11.0	703	1,848	50,302
Lake Washington Bridge, Seattle, Wash.[d]	11.0	971	1,583	46,350
South Capitol St. Bridge, Washington, D.C.	11.5	1,120	1,542	53,411

[a] Divided except as noted.
[b] Shoulder used as acceleration and deceleration lane; lane volume based on count of through lanes.
[c] No median divider.
[d] Unbalanced, 3/1.

a single occasion may have been an exceptional case with no other instance when the traffic volume equaled or even approached the reported values. It should be noted that many important capacity determinants, such as traffic composition, parking restrictions, and grades, are not available to assist in the evaluation of these data.

These maximum observed volumes are given primarily to acquaint the reader with the peak traffic that has been carried on some of the more heavily traveled routes. They are also intended to indicate the wide range of capacities of highways that are seemingly alike in type, but actually have significant differences in their physical, as well as traffic, characteristics. The reasons for much of the variation in capacity will become more apparent as the subject is developed in the succeeding material. However, considerable variation must still remain unexplained, awaiting further research.

The selection of the highways included in Tables 3.1 and 3.2, for two-way highways with two and three lanes, was based on the magnitude of the hourly traffic volume in

both directions of travel. The hourly traffic volume for the heavier direction of travel during the corresponding time period is, however, given in a separate column.

The selection of the highways to be included in Tables 3.3 through 3.6, for two-way highways with four or more lanes, was based on the magnitude of the traffic volume in the heavier direction of travel. The traffic volume for the lighter direction of travel during the corresponding time period is, however, given in a separate column. The selection of all one-way highways included in Tables 3.7 through 3.10 was based on the magnitude of the peak-hour traffic volume, given as the average number of vehicles per lane.

VOLUME CHARACTERISTICS

Spatial Variations in Traffic Flow

TRAFFIC DISTRIBUTION BY ROUTES

Figure 3.1 shows the breakdown by various average daily traffic flows of surfaced mileage on state primary systems in the

TABLE 3.4—HIGHEST REPORTED HOURLY VOLUMES ON FIVE-LANE, TWO-WAY HIGHWAYS IN THE UNITED STATES, 1961

| ROUTE AND LOCATION | AVERAGE LANE WIDTH (FT) | AVG. VOLUME (VPH/LANE) | | ADT FOR BOTH DIRECTIONS |
		LIGHT DIRECTION	HEAVY DIRECTION	
(a) MAJOR CITY STREETS				
Russell St., at Gwynns Falls Bridge, Baltimore, Md.[a]	10.0	921	890	—
E. Marginal Way, Seattle, Wash.[b]	10.0	902	879	44,000
Fourth Ave. S., Seattle, Wash.[b]	10.0	433	510	38,000
(b) BRIDGES AND TUNNELS				
Hackensack River Bridge, Secaucus, Hudson Co., N.J.[b]	10.0	1,418	1,463	65,000
First Avenue S., (Duwamish River Bridge), Seattle, Wash.[b]	10.0	502	1,102	37,500

[a] Three lanes reserved for heavier direction permanently.
[b] Center lane reversible, to operate three lanes in peak direction.

TABLE 3.5—HIGHEST REPORTED HOURLY VOLUMES ON SIX-LANE, TWO-WAY HIGHWAYS [a] IN THE UNITED STATES, 1961

ROUTE AND LOCATION	AVERAGE LANE WIDTH (FT)	AVG. VOLUME (VPH/LANE)		ADT FOR BOTH DIRECTIONS
		LIGHT DIRECTION	HEAVY DIRECTION	
(a) URBAN FREEWAYS				
Hollywood Freeway, at Highland, Los Angeles, Calif.	12.0	1,253	2,190	130,000
Eisenhower (Congress) Expressway, Chicago, Ill.	12.0	1,567	2,163	103,000
John Lodge Expressway, at Elmhurst, Detroit, Mich.	12.0	1,360	2,071	139,297
Schuylkill Expressway, Philadelphia, Pa.	12.0	1,370	2,015	113,291
Edsel Ford Expressway, at Russell-Rivard, Detroit, Mich.	12.0	—	1,925	132,554
(b) RURAL FREEWAYS				
U.S. 40, Delaware Mem. Br. Approach, Wilmington, Del.	12.0	708	733	28,909
I-35, S. of Austin, Travis Co., Tex.	12.0	74	396	7,170
U.S. 66, N.E. of Ill. 83, DuPage Co., Ill.	12.0	277	376	18,000
(c) URBAN EXPRESSWAYS AT GRADE				
U.S. 30 & 130, Pennsauken Twp., Camden Co., N.J.	10.0	703	1,340	69,114
Geo. M. Cohan Blvd., Providence, R.I.	10.0	875	1,331	55,900
Penrose Avenue, Philadelphia, Pa.	12.0	783	1,132	48,300
U.S. 99, S.W. Harbor Drive, Portland, Ore.	12.0	680	1,115	49,917
U.S. 1 & 401, Downtown Blvd., Raleigh, N.C.	11.0	440	847	32,500
(d) RURAL HIGHWAYS				
U.S. 46, Clifton, Passaic Co., N.J.	12.0	—	1,998	78,000
Rt. 90, Pearl Harbor Spur to Middle St., Honolulu, Haw.	11.0	411	995	37,671
U.S. 13-40, Farnhurst, Del.	14.0	806	989	60,000
U.S. 91-466, S. of Las Vegas, Nev.	10.6	372	683	35,888
(e) MAJOR CITY STREETS				
Alemany Blvd., San Francisco, Calif.	10.3	303	1,261	44,985
Elliot Avenue, Seattle, Wash.	11.0	393	1,155	43,500
Mich. 102, Base Line Road, Detroit, Mich.	12.0	763	1,060	59,000
Ala Moana, Honolulu, Haw.	11.0	553	1,035	47,640
Riverside Drive, Los Angeles, Calif.	11.7	431	991	53,000
(f) BRIDGES AND TUNNELS				
Aurora Ave. Bridge, Seattle, Wash.[b][c]	9.5	1,117	1,876	82,500
Memorial Bridge, Washington, D.C.[b][c]	9.3	1,188	1,722	68,590
Central Artery Tunnel, Boston, Mass.	12.0	1,226	1,685	75,462
San Fran.-Oakland Bay Bridge, Calif.[d]	9.7	1,171	1,533	—
Lincoln Tunnel, New York, N.Y.[b]	10.8	1,230	1,134	75,967

[a] Divided except as noted.
[b] Four lanes reserved for heavier direction during peak periods. Peak counts shown are for unbalanced operation; ADT for all conditions of operation.
[c] No median divider.
[d] Upper deck.

TABLE 3.6—HIGHEST REPORTED HOURLY VOLUMES ON EIGHT-LANE, TWO-WAY HIGHWAYS [a] IN THE UNITED STATES, 1961

ROUTE AND LOCATION	AVERAGE LANE WIDTH (FT)	AVG. VOLUME (VPH/LANE)		ADT FOR BOTH DIRECTIONS
		LIGHT DIRECTION	HEAVY DIRECTION	
(a) URBAN FREEWAYS				
Eisenhower Expressway, W. of Austin, Chicago, Ill.	12.0	1,445	2,155	164,000
Harbor Freeway, Los Angeles, Calif.	12.0	1,145	1,888	171,200
Hollywood Freeway, Los Angeles, Calif.	12.0	1,138	1,838	204,000
Pasadena Freeway, Los Angeles, Calif.	11.0	1,400	1,825	115,200
Bayshore Freeway, San Francisco, Calif.	12.0	1,538	1,798	137,000
(b) URBAN EXPRESSWAYS AT GRADE				
Lake Shore Drive, N. of LaSalle, Chicago, Ill.[b]	11.0	695	1,513	140,000
Rt. 92, Nimitz Highway, Honolulu, Haw.	12.0	257	864	52,226
(c) BRIDGES AND TUNNELS				
Santa Ana Freeway Bridge over Los Angeles River, Los Angeles, Calif.	12.0	1,412	1,725	189,000
George Washington Bridge, New York, N.Y.	10.8	1,179	1,554	106,247
(d) MAJOR CITY STREETS				
Wis. 190, W. Capitol Drive, Milwaukee, Wis.	9.0	417	693	47,954

[a] Divided.
[b] Six lanes reserved for heavier direction during peak periods. Count shown is for unbalanced operation; ADT for all conditions of operation.

TABLE 3.7—HIGHEST REPORTED HOURLY VOLUMES ON TWO-LANE, ONE-WAY HIGHWAYS IN THE UNITED STATES, 1961

ROUTE AND LOCATION	AVERAGE LANE WIDTH (FT)	AVG. VOLUME (VPH/LANE)	ADT
(a) MAJOR CITY STREETS			
Roosevelt Way N.E., Seattle, Wash.	12.5	1,238	15,500 [a]
Second Street, Tulsa, Okla.	13.0	967	12,036
U.S. 410, Main Street, Lewiston, Idaho	12.0	585	11,100
(b) BRIDGES AND TUNNELS			
L & N Bridge, Cincinnati, Ohio [b]	9.6	1,276	21,749
Marion Street Bridge, Salem, Ore.	12.0	672	10,500

[a] Weekday.
[b] Operates two-lane, one-way during peak periods. Count shown is during one-way operation; ADT includes all conditions of operation.

TABLE 3.8—HIGHEST REPORTED HOURLY VOLUMES ON THREE-LANE, ONE-WAY HIGHWAYS IN THE UNITED STATES, 1961

ROUTE AND LOCATION	AVERAGE LANE WIDTH (FT)	AVG. VOLUME (VPH/LANE)	ADT
(a) MAJOR CITY STREETS			
Fourth Avenue, Nashville, Tenn.	11.0	1,052	14,626
19th Avenue, Los Angeles, Calif.	11.0	849	18,000
11th Avenue, N.E., Seattle, Wash.	10.0	834	15,000
Western Avenue, Seattle, Wash.	10.0	658	12,100
Second Avenue, Spokane, Wash.	12.0	646	21,000
(b) BRIDGES AND TUNNELS			
Fourteenth St. Bridge (Southbound), Washington, D.C.	13.3	1,856	59,353
Martin Pena Bridge, P.R. 1, San Juan, P.R.	10.5	1,040	24,746

TABLE 3.9—HIGHEST REPORTED HOURLY VOLUMES ON FOUR-LANE, ONE-WAY HIGHWAYS IN THE UNITED STATES, 1961

ROUTE AND LOCATION	AVERAGE LANE WIDTH (FT)	AVG. VOLUME (VPH/LANE)	ADT
(a) MAJOR CITY STREETS			
Oak St., San Francisco, Calif.	10.7	958	18,910
Fell St., San Francisco, Calif.	10.0	816	19,284
12th St., Sacramento, Calif.[a]	11.0	750	63,600
Saginaw St., Lansing, Mich.	11.0	714	17,000
Michigan Avenue, Chicago, Ill.	12.0	653	24,000
(b) BRIDGES AND TUNNELS			
Spokane St. Bridge, Seattle, Wash.	10.5	1,402	56,000 [b]
Fourteenth St. Bridge (Northbound), Washington, D.C.	12.0	1,335	57,861

[a] One-half of a couplet; hourly volume given is for this street only; ADT is for both streets.
[b] Weekday.

TABLE 3.10—HIGHEST REPORTED HOURLY VOLUMES ON FIVE-LANE, ONE-WAY HIGHWAYS IN THE UNITED STATES, 1961

ROUTE AND LOCATION	AVERAGE LANE WIDTH (FT)	AVG. VOLUME (VPH/LANE)	ADT
(a) MAJOR CITY STREETS			
King Street, Honolulu, Haw.	10.0	619	30,000
Jefferson St., Phoenix, Ariz.	10.0	477	21,664

United States as of December 1962. Of a total of 446,391 miles of streets and highways reported in this classification, 44,881 miles or 10.1 percent were within municipal areas, while 401,510 miles or 89.9 percent were in rural areas.

When a comparison is made by traffic volumes, the significant differences between rural and urban highways are apparent. One-half of the rural mileage carries traffic volumes of less than 1,000 vehicles per day and 98 percent have volumes of less than 10,000 vehicles per day (vpd). On the other hand, traffic volumes on more than one-half of the municipal mileage exceed 4,000 vpd and on about one-fourth they exceed 10,000 vpd. Although comprising only 10 percent of the total mileage, municipal extensions account for 57.9 percent of the mileage where volumes exceed 10,000 vpd.

DIRECTIONAL DISTRIBUTION

On most two-way highways the annual average daily traffic has been found to be approximately the same in each direction. This is also the case for most 24-hr volumes, but holiday and weekend travel can cause an unbalanced total flow on specific days. The volume during any specific hour may, however, be much heavier in one direction than in the other. A knowledge of the traffic load in each direction for the peak periods of traffic flow is essential because of the critical effect an unbalanced flow can have on the needed capacity as related to the design and operation of a highway.

Typical directional distributions for urban and rural highways are given in Tables 3.1 through 3.6. For specific locations, directional distributions vary widely and the use of average values without confirming their applicability is not recommended. Even along one street or highway section, differing characteristics at various locations and traffic volume gains and losses at ingress and egress points may cause wide variations in directional distribution at different points.

LANE DISTRIBUTION

Where two or more lanes are available for travel in one direction, the number of vehicles in each lane may vary widely. The distribution of traffic between lanes for one

ANNUAL AVERAGE DAILY TRAFFIC

Figure 3.1. Percentage of surfaced mileage of state primary system in various volume groups, 1962.
(Source: Ref. *1*, 1962, Table SM-15)

direction of flow depends on several factors, including traffic volume, medial and marginal friction, proportion of slow moving vehicles, and the number and location of ingress and egress points. The origin and destination desires of the individual users are principal determinants for the distribution of vehicles between lanes near entrance or exit points. Outside the influence of these and other interferences adjacent to the traffic stream, lane distributions are influenced mainly by the variation in speeds and the volume of traffic.

Figure 3.2 shows average lane distributions in one direction found on selected six-lane freeways at various volumes. Chapter Eight, Ramps, contains a more detailed discussion of lane distributions under a variety of conditions.

TRAFFIC COMPOSITION

Variability also occurs in the percentage of the different types of vehicles comprising the traffic stream. The relative proportion

Figure 3.2. Effect of traffic volume on lane use
for six-lane facilities.
(Source: Ref. 21)

Figure 3.3. Traffic composition by time of day,
Wisconsin highways, weekdays, 1961.
(Source: Ref. 9)

of trucks and buses in the traffic stream has a significant effect on traffic speeds and other operating characteristics.

Of the 79,022,916 motor vehicles registered in the United States in 1962, 83.4 percent were passenger cars, 16.2 percent were trucks, and 0.4 percent were buses (*1*). The manner in which each type is used, however, creates wide variations in the relative proportion of each vehicle type in the traffic stream on a specific highway at any time. For example, Figure 3.3 shows the percentage of commercial vehicles in the total volume during various hours of the average weekday, as found in 1961 in Wisconsin.

Time Variations in Traffic Flow

Over the years, there has been a long-time trend in total vehicle-miles of travel that, except for the depression years (1932-1933) and the war years (1942-1944), has shown a consistent annual increase. Since 1950 this increase has been approximately 4.6 percent compounded annually (*1*).

Within the annual increase in motor vehicle travel, there exist certain cyclical variations with respect to time. The major variations may be expressed as seasonal, weekly, and daily time patterns of traffic flow. Peaking characteristics within the peak hours also should be considered, although few data are available on the nature and pattern of repetition. Because of their importance in volume and capacity determinations, the general characteristics of these time patterns are illustrated.

SEASONAL VARIATIONS

The seasonal pattern of traffic volume on any highway is closely related to economic and social demands for transportation. A few representative patterns are shown in Figure 3.4. A typical variation for rural highways influenced by summer recreational traffic is shown using 1961 data from ten permanent counting stations in the State of Washington. Variations in urban seasonal patterns tend to be much less pronounced, as indicated by the patterns for the average of six urban stations in the State of Washington (1961) and for a three-year average of five control stations in Tucson, Ariz. (1958-60). The variation in seasonal patterns, because of climate, is also apparent in Figure 3.4 when the two urban patterns are compared. It should be noted that for these highways the May and October volumes are close to the annual average.

WEEKLY VARIATIONS

Figure 3.5 shows the characteristic weekly patterns of traffic volume for three area types: urban Nashville, Tenn.; the periphery of Lexington, Ky.; and rural highways in Mississippi. Although individual locations will vary greatly, certain general character-

Four-lane urban arterial congested with traffic also has an extremely high percentage of trucks in direction away from camera. Note median barrier.
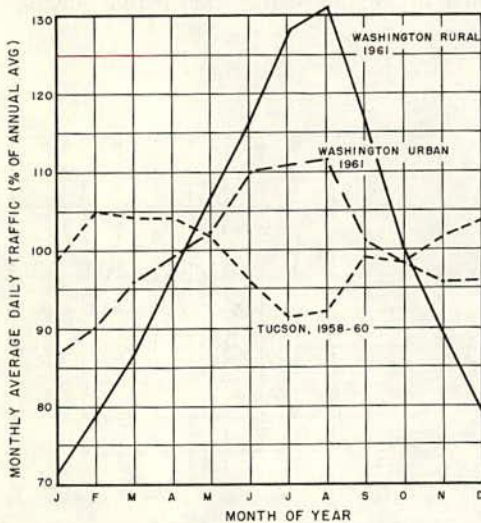
Figure 3.4. Examples of monthly traffic volume variations.
(Sources: Washington State Dept. of Highways and Tucson, Ariz., Area Transp. Study)
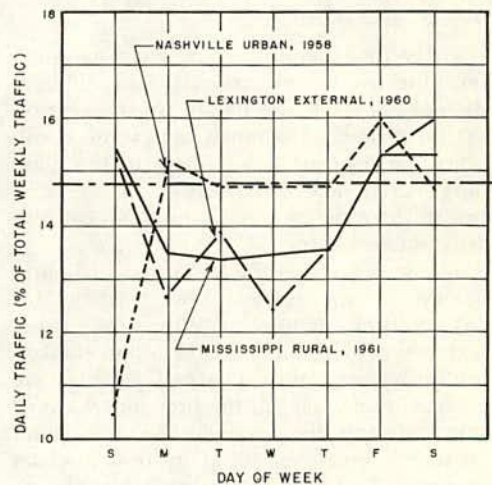
Figure 3.5. Examples of weekly traffic variations.
(Sources: Mississippi State Highway Dept.; Nashville, Tenn., Metro. Area Transp. Study; Lexington-Fayette Co., Ky., Transp. Study)
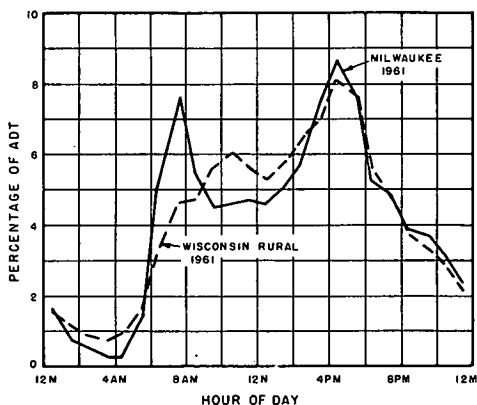
Figure 3.6. Hourly variations of traffic for average weekday.
(Source: Ref. 9)

istics are portrayed in the figure. On urban streets and highways, the Monday through Friday daily traffic volume is fairly stable, and the Sunday volume quite low. The reverse is generally true on rural highways, with peak daily volumes occurring on summer Sundays and holidays. Individual locations may have characteristics different from any of the patterns shown.

DAILY VARIATIONS

Daily time patterns show wide variation for different routes, and also for different days of the week and months of the year on the same route. Inasmuch as peak-hour volumes are a prime determinant in the planning, design and operation of highways, a few of the more characteristic daily patterns are discussed here.

Figure 3.6 shows the variations of hourly volume for the average 1961 weekday for (a) 49 rural stations on Wisconsin's trunk highway system, and (b) 34 urban stations in Milwaukee, Wis. The two patterns are similar in one respect, the preponderance of travel during the daylight hours. About 70 to 75 percent of the daily travel occurs in the 12-hr period from 7:00 AM to 7:00 PM. The Milwaukee stations depict a pattern of two distinct daytime peaks resulting from the repetitive travel from home to centers of employment, business,

and commerce, and return. The rural stations exhibit one pronounced peak in the late afternoon. The magnitude and duration of the maximum volumes vary considerably between routes and within a given urban area. Seasonal patterns and special traffic generators may create wide variations on any specific route.

Figure 3.7 shows two daily patterns in August 1961 for the Calumet Expressway, a radial freeway in south suburban Chicago, Ill., which, at this location, is 4-lane. Here, the weekday and Sunday patterns are distinctly different. The weekday pattern is dominated by travel to and from the central city, whereas the Sunday pattern presumably reflects recreational travel.

The previous discussion was concerned with total two-way volumes on the highways studied. When one-way traffic is considered separately, the volume curve may tend to have a single peak period of greater magnitude than when the total two-way flow is considered.

VARIATIONS WITHIN THE HOUR (PEAKING CHARACTERISTICS)

Before going into more detail on the significance of hourly volumes as a determinant in the planning, design, and operation of highways, the short-period fluctua-
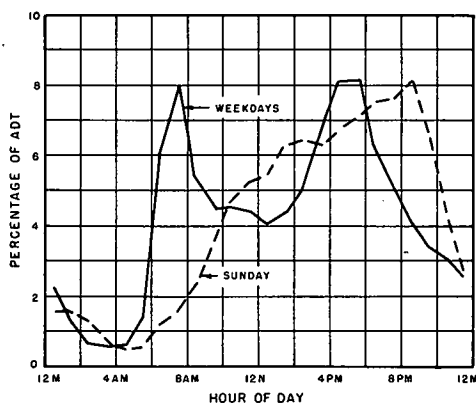
Figure 3.7. Hourly variations of daily traffic on Calumet Expressway, 1961.
(Source: Traffic Characteristics on Illinois Highways, 1961)

Slower vehicle forces following vehicles to reduce their speeds; queuing effect commencing on two-lane rural highway.

tions in traffic flows within the hour should be considered. Although hourly volumes are normally used in planning and design, the ability of a highway to accommodate satisfactorily an hourly volume depends primarily on the magnitude and sequence of these short-period fluctuations.

No matter what criteria are used for the design and operation of a highway, it is necessary to know what the nature of the traffic demand will be, as well as its specific value. A peak-hour volume does not necessarily imply that a high rate of flow will exist for less than a full hour, more than an hour, or approximately one hour; it is simply an estimate of the maximum number of vehicles expected on a facility during a full 60-min period. Due to the nature of the peak-hour demand and the statistically variable nature of traffic, it is known that short-term rates of flow within the peak hour are often quite variable.

The statistical variability of volumes of traffic is affected by the time period involved. As the time period is reduced, the average number of vehicles for that time period will reduce accordingly. For example, if the average hourly volume were 1,800 vph, the average minute volume would be 30 vpm and the average second volume would

be $\frac{1}{2}$ vps, based on the hourly volume. The variability of smaller mean values is greater than that of larger mean values, when expressed as a percentage of the mean. The narrowing of the confidence interval band for increasing mean values is characteristic not only of the Poisson distribution, which closely approximates the distribution of light volumes of traffic, but also of the many other distributions which have been used to approximate various actual traffic distributions. Even the normal distribution exhibits these same characteristics, although it is rarely used as an example of existing traffic distributions. Thus, even without the occurrence of a change of volume within a given peak hour, a short-term period within this hour has increased probability of exceeding its mean by a given percent than does the whole hour.

For planning purposes, future volumes are usually estimated for the peak-hour period. In order to relate such volumes to a design peak rate of flow, the factors which affect this relationship must be established and evaluated.

Various studies, notably those concerned with freeway, tunnel, and intersection operation, have been concerned with this relationship of peak periods to total peak-hourly
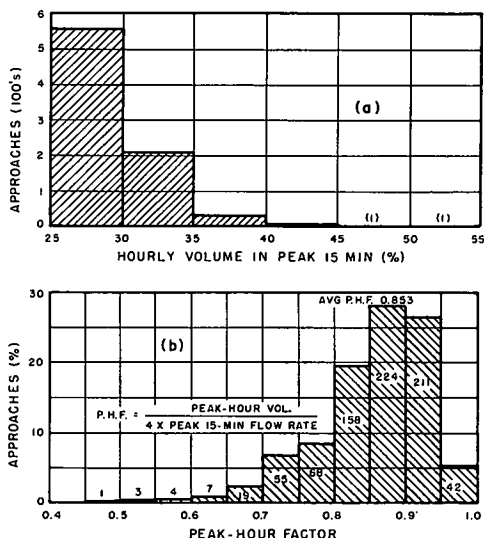
*Figure 3.8. Distribution of (a) magnitude of 15-min peak flow rate and (b) 15-min peak-hour factors * at 792 signalized urban intersections.*
(Source: BPR 1955-56 studies, and Ref. 2)

volume. Variations within other than the peak hour are usually less critical and seldom require investigation. Variations within the peak hour on ordinary two-lane and multilane highways without access control have received relatively little study as yet.

A Michigan study of seven different multilane highways of various types found that the peak 15 min carried from 26.1 to 30.7 percent of the peak-hour traffic, with an average of 28.5 percent (3).

Figure 3.8 shows the distribution of peak 15-min rates of flow as a percent of hourly volume computed for 792 signalized intersection approaches on all types of streets. The peak 15 min contained from 25 to 55 percent of the total hourly volume, averaging 29.3 percent for all approaches.

Freeway and expressway studies have frequently used 5- or 6-min flows as being more indicative than hourly volumes. Figure 3.9 shows the relationship of peak 5-min flows to peak-hour volumes, summarized from data for 225 study locations on free-

* See Chapter Six.

ways in 54 cities as reported to the Bureau of Public Roads (4). Here, size of the metropolitan area, as measured by population, is used as a primary criterion. Although a reduction in the peaking effect with increasing city size is noted, the wide scatter of individual readings shown by the standard deviation suggests the need for caution in applying this relationship at a specific location. The reduced peaking on those highways reported as non-free-flowing during the peak hour points out the damping effect of congestion..

Another study utilized data from more than 200 freeway traffic studies, including work by the Texas Highway Department, the previously mentioned work of the Bureau of Public Roads, studies conducted by the Texas Transportation Institute, and specific studies conducted for the project (5). The relationship between short-period (5-min) traffic flow to total hourly flow was determined from these data.

Consideration of all factors likely to influence peaking characteristics was not possible in this study, however. For instance, the degree of utilization, or "loading," of a freeway in some instances is controlled by the capacity and operation of the supporting street system and that capacity
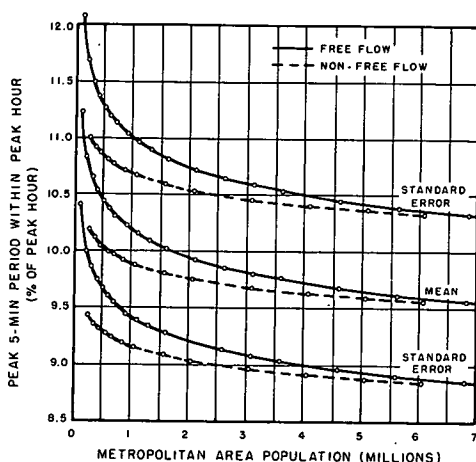
Figure 3.9. Peaking trends related to population and quality of traffic flow.
(Source: Ref. 4)
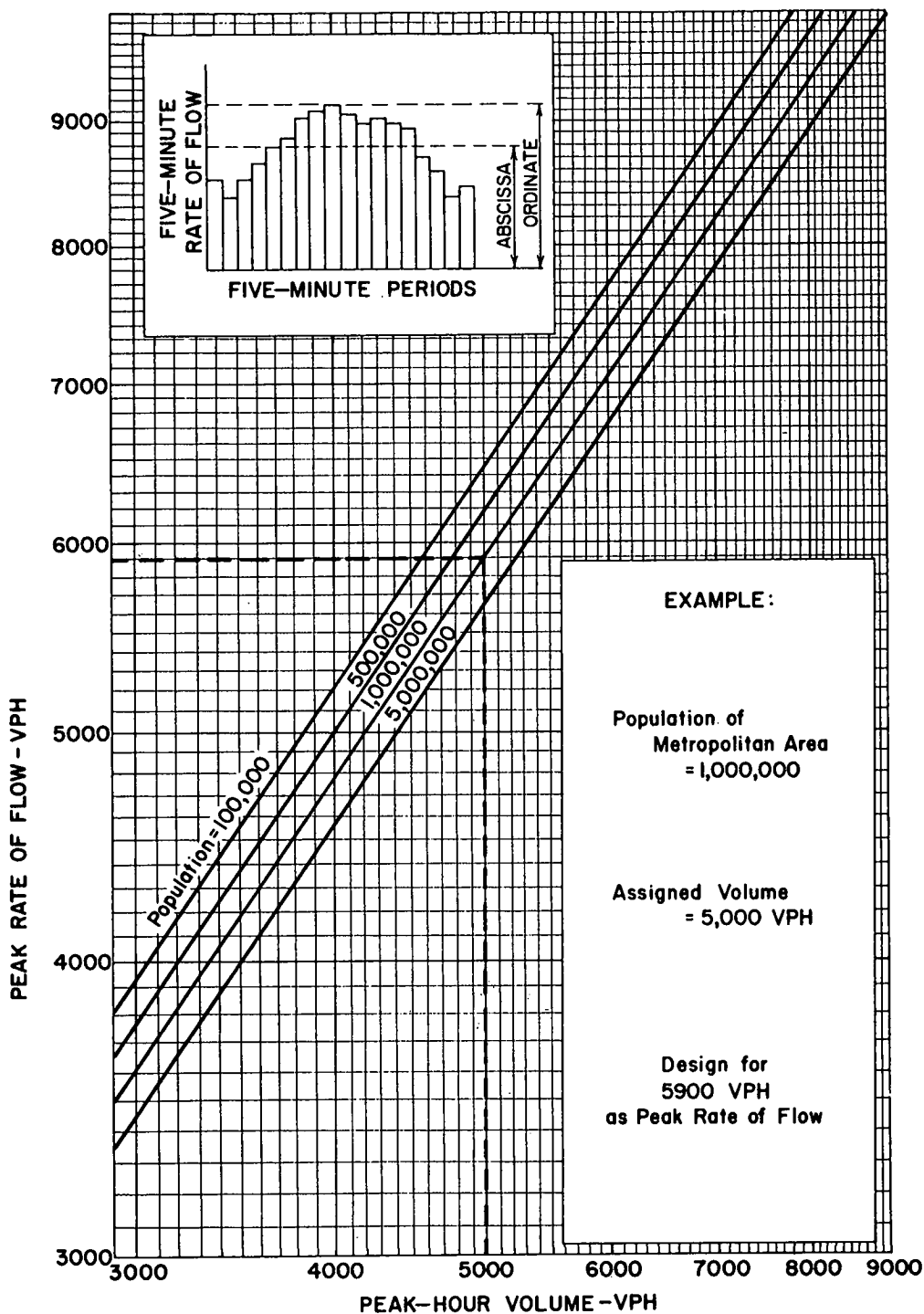
Figure 3.10. *Determination of rate of flow for highest 5-min interval from rate of flow for the whole peak hour.*
(Source: Ref. 5)
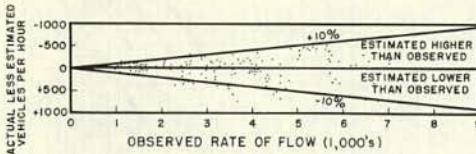
Figure 3.11. Relationship between error in estimated rate of flow and observed rate of flow.
(Source: Ref. 5)

also sometimes limits "unloading," which results in impaired freeway operation. There was not sufficient knowledge of each of the freeways, except those in Texas and a few other specific sites, to permit consideration of these characteristics. However, congestion was not apparent in the immediate vicinity of any of the study sites. It is possible that much better correlations of the results would have been possible had all conditions been known. Those freeways known to have good "loading" and "unloading" characteristics showed very good correlation of the data.

Many characteristics related to trip generation—such as geographical and time concentrations of trips, character of the freeway (radial, circumferential, etc.), character of the supporting street system, population, area served—have marked effects on peaking characteristics. However, as was the case in the Bureau of Public Roads study, it was possible from the data available to study only the relationship of peaking to the population of the city or urban area. The results (Fig. 3.10) are based on the data for 132 peak periods from studies in

31 cities in 18 states. The variables are statistically significant and the curves fit the available data with a standard deviation of 5 percent. They are also in reasonable conformance with the results shown in Figure 3.9.

Figure 3.11 shows the relationship between estimated rates of flow and observed rates of flow and includes a 10 percent error band within which most points were included. Figure 3.12 shows the frequency distribution of the percent error involved in using Figure 3.10 to estimate the peak rates of flow. As can be noted, the errors are somewhat normally distributed.

Overtaking and passing maneuver on rural low-volume freeway.

Relating Hourly Volumes to Annual Averages and Peak Flows

The foregoing discussion has served to indicate that normal traffic patterns develop peaking characteristics with respect to both time and location. Because these variations in traffic flow represent patterns of travel desire, the adequacy of a highway cannot be judged by its ability to carry the average volume, but rather must be evaluated in terms of its ability to function properly under specified peak loads. This concept that the capacity of a highway is a function of both the physical features of the roadway and the pattern of demand shapes present highway practices.

Figure 3.12. Frequency distribution of percent difference between estimated and observed rates of flow.
(Source: Ref. 5)

In subsequent chapters, traffic volumes are expressed in terms of vehicles per hour. However, the engineer often finds that complete and detailed hourly volume data are not available. Many times the only information may be a group of scattered counts or an estimate of annual average daily traffic (AADT), based on counts made at intervals throughout the year on the highway under consideration or on similar highways. In such cases, a method for adjusting the available counts to determine the hourly capacity necessary becomes a matter of paramount importance. A clear understanding of the variations in traffic load that may be expected is essential in this determination. Without this knowledge the application of traffic count data to planning, design, and operation cannot be completely successful.

DETERMINATION OF PEAK HOURLY VOLUMES

Closely related to the fluctuations in traffic flow is the selection of the specific hourly volume which should be used for design purposes or which should be established as the reasonable volume which an existing street or highway should be expected to accommodate. Therefore, knowledge of those brief, but frequently repeated, peak volumes is essential.

When hourly traffic counts for a full year are available for a highway under consideration, it is possible to show the distribution of hourly volumes by arranging these volumes in descending order of magnitude. These volumes can be shown either as a continuous array (Fig. 3.13) or as an accumulation of the total vehicles served at or above various volume levels (Fig. 3.14). In the example shown, one-half of the hours carry volumes of less than 235 vph, but only 13 percent of the total annual volume is served during those hours. Conversely, one-half of the traffic is served in the less than 20 percent of hours which carry over 425 vph. It is obvious that a highway designed to give an acceptable level of service on the basis of either of these hourly volumes would be less than adequate on many occasions when higher demand existed. On the other hand, a highway designed to provide a high level of service for the maxi-
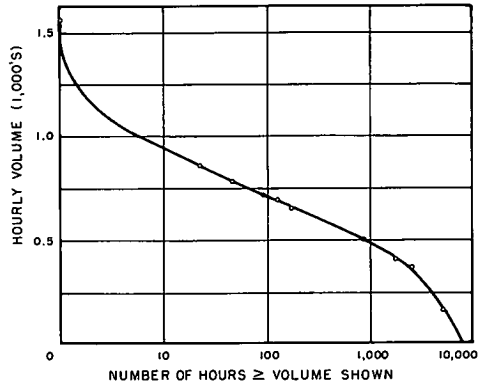
Figure 3.13. Yearly variation of hourly traffic volumes in descending order of magnitude.
(Source: Individual ATR station; analyzed by BPR)

mum recorded hourly volume of 1,575 vph would have substantial excess capacity during all but one hour of the year, an economically unfeasible situation. The selection of an appropriate value as the hourly volume to be served is, thus, a compromise between annual service provided and cost. Customary practice in the United States would base design on a value between the 10th and 50th highest hourly volume or, in this example, between three and four times the
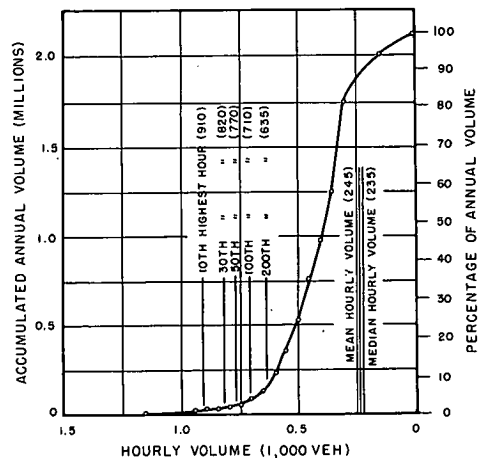
Figure 3.14. Amount and percentage of annual volume served at or above hourly volumes shown.
(Source: Figure 3.13)

average hourly volume. Frequently, the 30th highest hourly volume is used, but it is not a rigid criterion.

Table 3.11 gives the yearly traffic patterns for two rural highways in the same state. For these two roads, neither the annual volume nor the maximum hourly volume, by itself, adequately describes operating conditions; both measures must be considered. Both highways have an annual average volume of approximately 7,200 vpd; yet the peak hourly volumes on Road A are much higher than those on Road B due to greater fluctuation in traffic flow. The volume during the peak hour on Road A is 2.5 times that of Road B. Road A has volumes exceeding 900 vph 5.7 percent of the time, whereas on Road B this volume is exceeded only 0.1 percent of the time.

It is apparent that if peak volumes are to be handled adequately Road A would require a higher-type design than Road B, even though the AADT's are the same.

Figures 3.15 and 3.16 show the average

TABLE 3.11—OBSERVED HOURLY TRAFFIC VOLUMES ON TWO RURAL HIGHWAY SECTIONS WITH IDENTICAL AVERAGE DAILY TRAFFIC

ITEM	ROAD A	ROAD B
AADT	7,200	7,200
Maximum hourly volume	2,462	988
10th highest hourly volume	2,106	896
20th highest hourly volume	1,986	880
30th highest hourly volume	1,892	864
50th highest hourly volume	1,720	840
100th highest hourly volume	1,506	800
200th highest hourly volume	1,270	762
400th highest hourly volume	1,010	644
600th highest hourly volume	824	588

yearly traffic patterns for one direction and two directions and for urban and rural highways from a sample of 113 continuous counting stations in 17 states. The curves
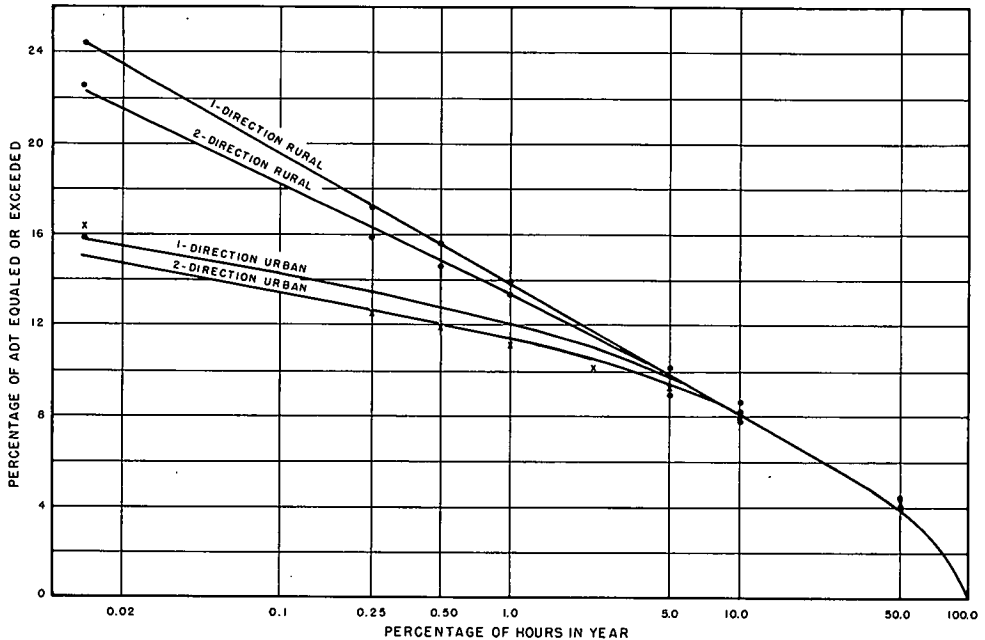
Figure 3.15. Percentage of ADT recorded during all hours of the year on 113 selected urban and rural roads, 1959-1960.
(Source: BPR)
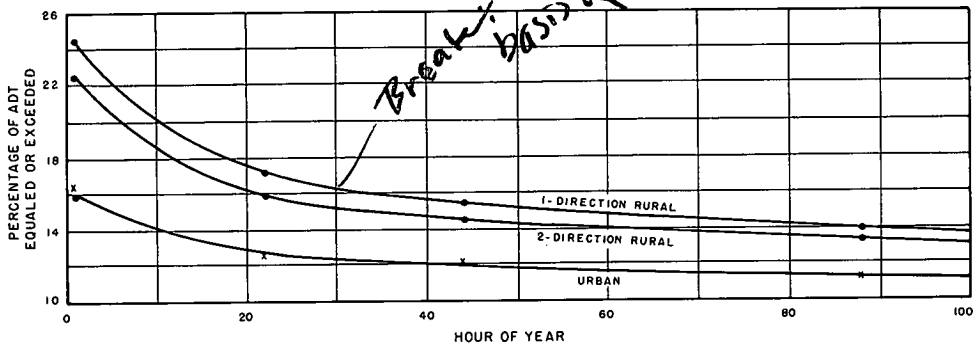
Figure 3.16. ADT recorded during 100 highest hours of year on 113 selected urban and rural roads, 1959-60.
(Source: BPR)

represent the median values for highways ranging in volume from 429 to 66,624 AADT.

Looking first at all curves, a similar distribution of traffic volumes is observed throughout the lower 90 percent of the hours carrying from 0 to 8 percent of the AADT. About three-fourths of the total annual traffic is carried in these hours. For the 10 percent of hours of highest volume, the data in all four classes are best represented mathematically by exponential curves of the order $y = a + b^x$. Thus plotted in semilogarithmic form (Fig. 3.15), no demonstrated break in the curves is apparent at the 30th highest hour. Plotted directly, however, such a break is evident (Fig. 3.16).

Consider next the differences in rural and urban patterns. Although there are specific exceptions to the generalization, it can be said that peak traffic volumes in urban areas are a somewhat lower percentage of the AADT than on rural highways. One reason for this pattern is discussed in the preceding section on time variations of traffic flow. In general, urban highways are less affected by seasonal, weekly, and daily variations in travel demand, with traffic being distributed more uniformly throughout the time period considered. Traffic variations become less pronounced as the types of traffic (composition, trip length, purpose) using the highway become more varied. The extent of development in the area traversed by the highway is a major factor.

Figures 3.15 and 3.16 show the difference in peak-hour percentage between one-directional and two-directional volumes. The volume variations in one direction have been expressed as a percentage of the one-direction AADT, as found on the sample of highways of more than two lanes, whereas the variations in two directions combined are expressed as percentages of the two-direction AADT. It can be seen that for the average condition the peaking characteristic for one direction of flow is substantially higher than that for two directions.

All measurable continuous volume patterns must, of course, be obtained from specific locations on existing highways, and measure variations at those particular points. At such points, the maximum volume recorded will be the lesser of two values—either the peak demand for use or the capacity of the highway, whichever governs. Unless observations are from highways with excess capacity during all hours, the effect of capacity limitations on the magnitude and duration of peak traffic flows will be a significant, but unquantified, factor affecting time-volume relationships.

RELATION OF HOURLY VOLUMES TO ANNUAL AVERAGE DAILY TRAFFIC

Continuous volume counts are available at only a limited number of locations on the existing highway system, and then only depict past occurrences. The most common measure used in reporting the traffic on a

TABLE 3.12—PERCENTAGE OF AADT IN PEAK HOUR FOR ONE DIRECTION
AND BOTH DIRECTIONS BY PEAK HOUR, 30TH HIGHEST HOUR,
AND 200TH HIGHEST HOUR, BY TYPE OF FACILITY·

| TYPE OF FACILITY | PERCENTAGE OF AADT IN PEAK HOURS | | | | | |
| | ONE DIRECTION | | | BOTH DIRECTIONS | | |
	PEAK HOUR	30TH HIGHEST HOUR	200TH HIGHEST HOUR	PEAK HOUR	30TH HIGHEST HOUR	200TH HIGHEST HOUR
Rural:						
Freeway	23.6	15.4	11.4	18.3	13.5	10.9
Expressway	21.5	14.1	10.6	19.2	12.7	9.7
Highway with more than 2 lanes	21.2	13.7	10.3	16.4	12.7	9.9
2-Lane two-way highway	—	—	—	19.7	13.6	11.2
Urban:						
Freeway	15.0	12.7	10.7	13.6	11.0	9.6
Expressway	14.6	11.4	8.9	11.6	9.5	8.3
Street with more than 2 lanes	13.8	11.1	9.6	12.0	10.0	8.7
2-Lane two-way street	—	—	—	13.4	10.6	9.0

highway is the annual average daily traffic flow (AADT). For the most part, the AADT of a given roadway is estimated from short counts adjusted by factors obtained at one or more continuous counting stations having assumed similar time patterns.

As a guide in relating annual traffic volumes and peak-hourly flows, an analysis of traffic data collected during 1961 and 1962 throughout the United States is presented in Appendix A of this manual, by region, by rural and urban location, and by various design types (freeways, expressways, highways of more than two lanes, two-lane highways). Average and maximum 24-hr volumes are given and, in accordance with present practice, various categories of hourly volumes are expressed as a percentage of the AADT. An examination of these data, which are summarized in Table 3.12 and Figures 3.17 through 3.20, leads to the following general conclusions:

1. There is a wide variation in the percentage of AADT in the highest hour within each class of highway.

2. The variation decreases from the highest hour through the 200th highest hour, and decreases as AADT increases.

3. The percentage of AADT in the highest hours is generally less for urban highways than for rural highways and is less for two-directional traffic than for one-directional traffic.

4. The percentage of AADT in the highest hours is generally greater for highways with very low AADT (under 1,000).

5. The percentage of AADT in an hour typically decreases from the highest hour through the 200th highest hour in a smooth curve, without a sudden change in the rate of change at any point.

The foregoing discussion emphasizes the problem of selecting a measured or predicted traffic volume to be used for design purposes. That this volume should be greater than an average volume is undeniable. The selection of an appropriate design hourly volume becomes an economic and system consideration of balancing expected benefits with construction costs; this involves administrative decisions beyond the scope of this manual.

TRENDS IN HIGHEST HOUR RELATIONSHIPS

In this section, relationships of the 30th highest hour will be discussed. This is be-
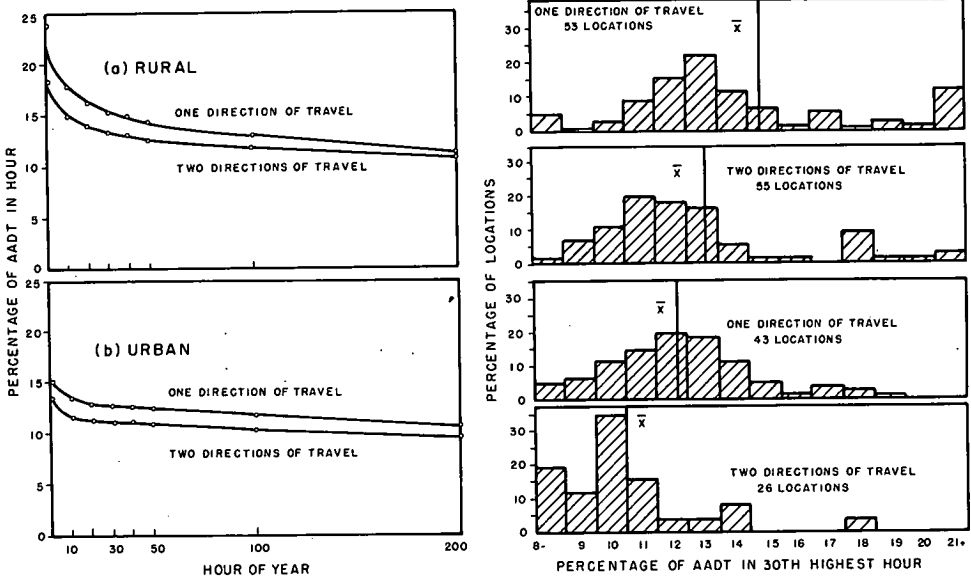
Figure 3.17. Relation of hourly volumes and annual average daily traffic on freeways.·
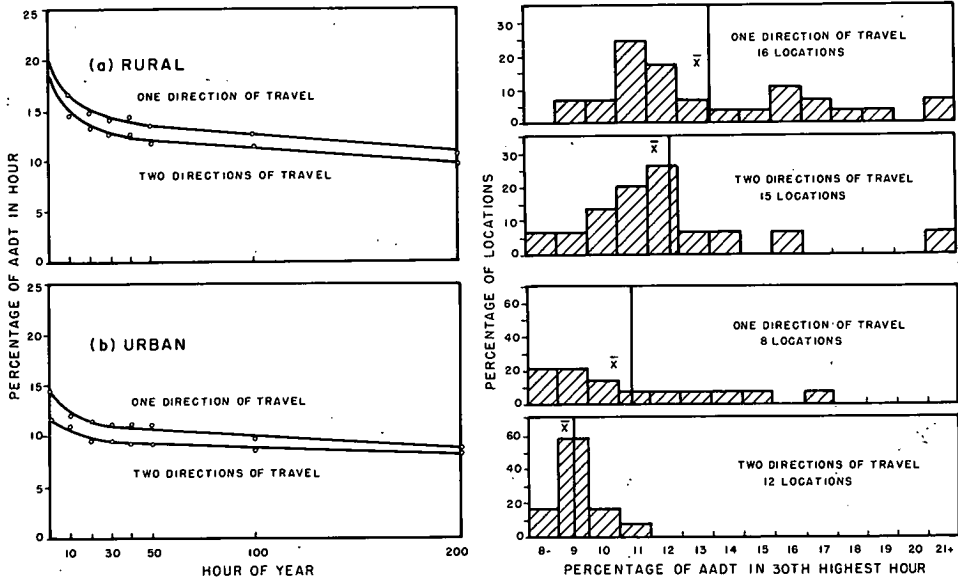
Figure 3.18. Relation of hourly volumes and annual average daily traffic on expressways.
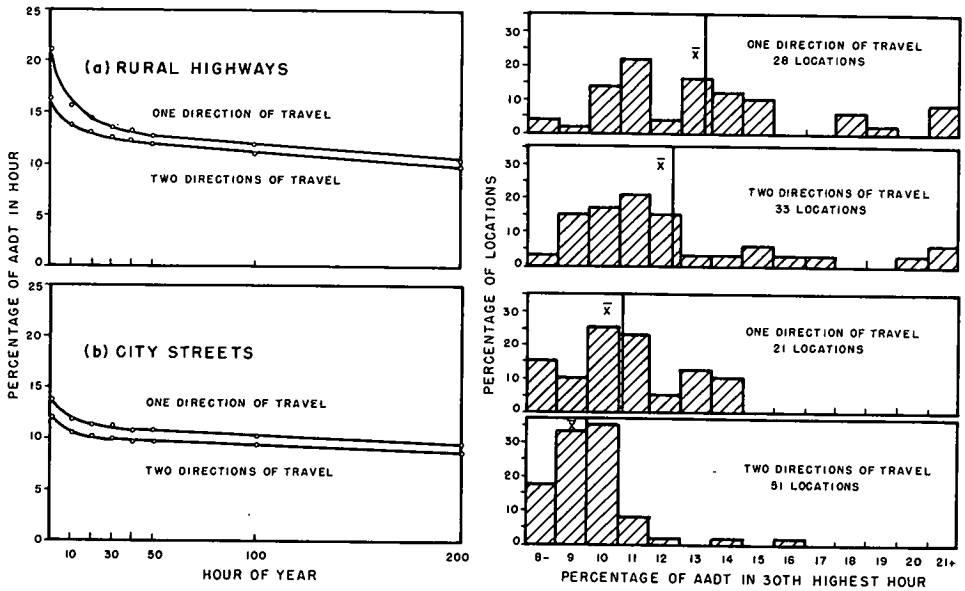
Figure 3.19. Relation of hourly volumes and annual average daily traffic on ordinary multilane highways.
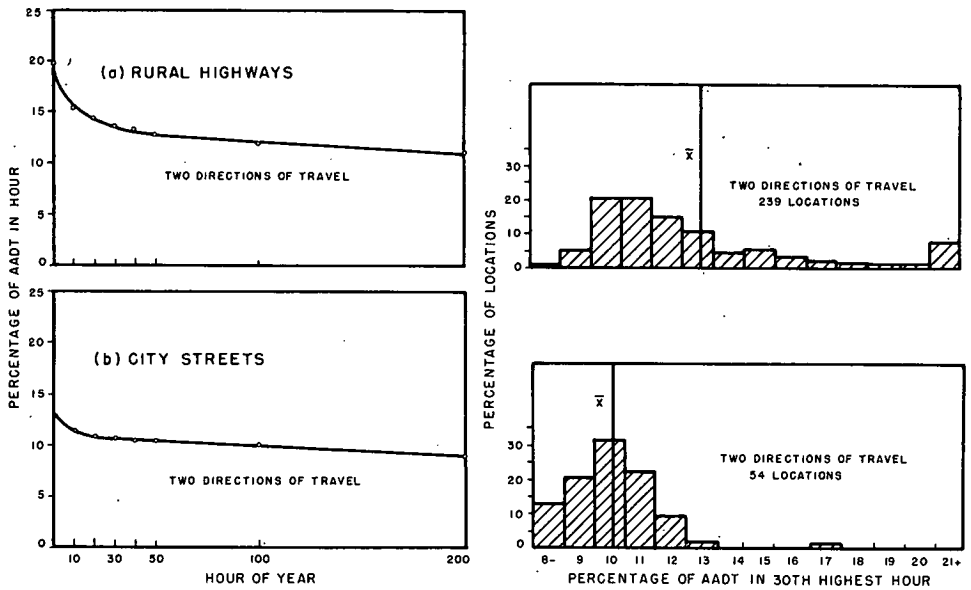
Figure 3.20. Relation of hourly volumes and annual average daily traffic on two-lane highways.

Traffic signals such as this special pedestrian one interrupt vehicular traffic to provide for crossing movement.

cause many states utilize the 30th highest hour as a design criterion for rural highways, thus data on the 30th highest hour relationships are readily available. This frequent reference to the 30th highest hour should not be misconstrued as a recommendation for rigid adoption, but rather as an example of typical highest hour relationships and trends. These same relationships and trends are generally true for other high-volume hours within the normal design range.

Early investigations in the area of 30th highest hour trends found little, if any, change in the ratio of the 30th highest hour to AADT from year to year. It was thus concluded that an increase in AADT was followed by a corresponding increase in the high hourly volumes on any particular highway. More recent investigations, based on extended coverage and longer periods of continuous-count operation, have not confirmed the above conclusion (6, 7, 8). The findings of three of the more significant studies, based on 30th highest hour relationships, are as follows:

1. The 30th highest hour factor generally decreases as the AADT on a highway increases.

2. The reduction rate for the higher 30th highest hour factors is much greater than for low factors.

3. Highways through areas of low population or sparse development, or those subject to high seasonal traffic fluctuations, have high 30th highest hour factors. Development or growth in the surrounding area tends to lower these factors more rapidly than would otherwise occur.

Under light volumes the excess capacity of a particular highway allows unimpeded travel whenever desired, whereas limited trip purposes in sparsely settled areas create short but intense periods of travel demand. This creates very high factors. As total volume increases and/or the surrounding area develops, an increase in trip purposes tends to spread the travel demand throughout the day more evenly. At some time, capacity limitations of the highway itself may

TABLE 3.13—AVERAGE SPEED, BY
VEHICLE TYPE, 1946-1964

| | AVERAGE SPEED (MPH) | | |
YEAR	PAS-SENGER CARS	BUSES	TRUCKS	ALL VE-HICLES
1946	46.1	47.8	40.2	45.2
1947	48.1	48.4	42.5	46.9
1948	48.8	50.0	43.1	47.7
1949	48.7	50.3	43.5	47.6
1950	48.7	49.8	43.0	47.6
1951	50.1	51.2	44.4	48.9
1952	50.8	52.1	45.0	49.5
1953	51.3	51.8	45.1	49.8
1954	51.4	51.8	45.4	50.0
1955	52.0	52.3	45.6	50.5
1956	52.0	52.2	46.2	50.6
1957	52.6	52.6	47.0	51.4
1958	52.8	53.6	47.3	51.7
1959	53.2	53.5	47.3	51.9
1960	53.8	55.5	48.2	52.6
1961	53.7	55.3	48.3	52.6
1962	55.1	56.0	49.4	53.8
1963	57.1	58.1	51.3	55.8
1964 [a]	56.9	57.3	50.9	55.6

SOURCE: Bureau of Public Roads, "Speed Trends," various years.
[a] Preliminary.

An analysis of 1947 to 1961 data from continuous traffic counting stations on rural highways in Wisconsin showed that the decrease in the 30th highest hour factor within any group of stations with similar traffic patterns was mainly a function of an increase in AADT (6). Another study in New Jersey of data from 69 stations indicated that the decrease in the 30th highest hour factor was principally a function of time, decreasing at an average rate of 2.3 percent compounded per year (7). A third study of 160 stations scattered throughout the United States found the relationship to be a function of both volume increase and time (8).

Each study represents an analysis of data limited to specific states or route types; hence, the findings cannot be assumed applicable for other than the localities studied. Nevertheless, they represent a methodology that may be used to estimate or forecast peak traffic flow more accurately. Further research is necessary to define more adequately the relative effects of these and other factors on the trend in peak rates of traffic flow.

SPEED CHARACTERISTICS

No discussion of highway or street capacity would be complete without consideration of the operating speed under the given conditions. Much of the driver's evaluation of a highway depends on the speed at which he can operate. The engineers' definition of service volumes is also predicated in part on the relationships between volume and speed on any given highway.

Given a highway on which vehicles can be operated at 70 mph if volume levels are low, some lesser speed is usually tolerated by the road user under many conditions.

Current experience on rural highways of advanced design indicates that more than 95 percent of the drivers do not exceed speeds of 75 to 80 mph, and appear satisfied when geometric and traffic situations result in average operating speeds between 55 and 65 mph. Unfortunately, no equivalent data are available for urban streets. Whether or not it is necessary or desirable to provide identical operating conditions for urban as for rural highways remains subject to dis-

limit the amount of traffic that can be carried during the peak hours, either suppressing travel or lengthening the time over which the peak periods extend. Therefore, a decrease in any high hour factor does not imply a decrease in the volume during peak periods, but only a larger percentage increase in total traffic, especially during the off-peak hours. The factors, therefore, are subject to the same considerations as described in the section on time variations in traffic flow, where peak periods of flow are a smaller percentage of the total under high volume conditions and in well-developed areas. Under extreme conditions, any traffic growth must take place in off-peak periods due to congestion during the peak periods.

The magnitude of change in the 30th highest hour or any other level selected will depend on many factors and should be determined for each state, area, or, if possible, specific highway. Initial studies in this field have already been undertaken.

cussion, but at present it is usually assumed that drivers will accept somewhat poorer service in urban areas than in rural.

Speed Trends

Although any given set of speed observations may be influenced by such items as volume, capacity, design, weather, or traffic control devices, the long-range speed trend has been gradually increasing, as shown in Table 3.13, which gives the result of speed studies conducted by more than one-half of the states on level tangent sections of main rural highways, during periods of relatively low traffic densities when most drivers can travel at their desired speeds, subject, of course, to applicable speed limits and enforcement levels. Measurements were made on both two-lane and multilane facilities, many of which were not of advanced design.

Some measure of the effect of modern design on the speed trends can be noted from a 1961 traffic speed study by the Wisconsin State Highway Commission (9). During 1961 the average speed of vehicles observed during low-volume periods on level tangent sections of the regular rural state highway system was 53.2 mph. Under similar conditions, the average speed on the completed portions of the Interstate highway system was 60.1 mph, a reflection of the influence of better design. This finding is typical of those being made by the many states which study traffic speed trends.

The speed trends shown in Table 3.13, as noted, are based on observations on a selected set of highways at times when drivers can travel at desired speeds. Thus, they indicate an increase in desired speeds and, to a lesser extent, the influence of improved highway design. But speed studies made in this manner may not reflect the impact of capacity limitations on speeds, so that a series of speed measurements made during hours of peak flow rates may have a long-range trend of decreasing speed. As time passes the capacity of an open rural highway remains relatively constant, but the number of hours during which the volume rate approaches capacity usually increases. Under these conditions there will be more hours when vehicles will not be able to drive at their desired speed and the average speed
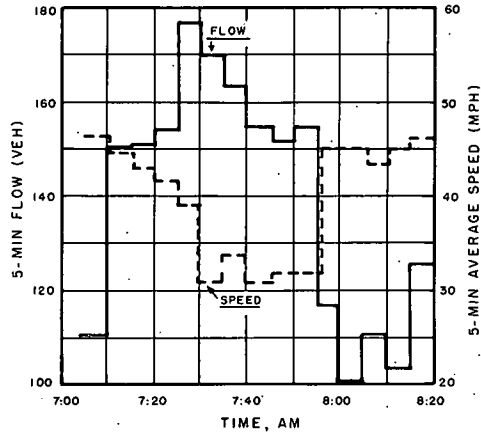
Figure 3.21. Variation of flow and speed during 5-min intervals of morning peak period, Gulf Freeway, Houston, Tex.
(Source: Ref. 10)

for that particular segment of highway will tend to decrease with time.

Although the effects described in the foregoing are found on all types of highways, they are most clear-cut on freeways and expressways, where few roadside frictions are present to produce other adverse effects. Hence, in most of the remainder of this discussion of speed characteristics, examples taken from freeway operations are employed.

Daily Speed Variations

The previously noted influences of volume and capacity on speeds can be observed when volume or density curves are superimposed on the speed curves for like time periods on the same highway. Typically, a speed reduction is found with increasing volume. Figure 3.21 shows this relationship during the morning rush hour for the median lane of the Gulf Freeway in Houston, Tex., for 5-min increments.

A similar relationship over a 12-hr period is shown in Fig 3.22, derived from data for all lanes in one direction on the Edsel Ford Expressway in Detroit, Mich. The minimum speed during the period occurred during the morning peak, just prior to 8:00 AM, and corresponds to the hour of peak flow rate.
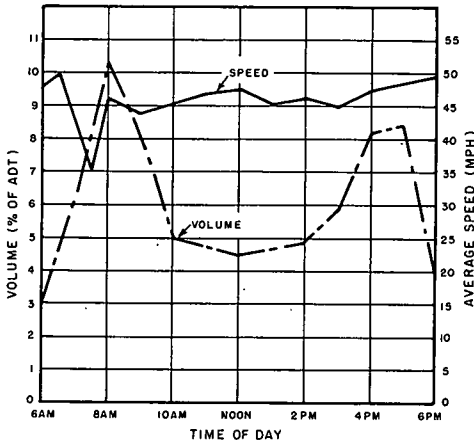
Figure 3.22. Variation in volume and speed by time of day, Ford Expressway, Detroit, 1959. (Source: Ref. 3)

In this example, it is important to note that in the afternoon peak a lesser volume increase did not result in a speed reduction; instead, it was accompanied by a slight *increase* in average speed. This phenomenon, observed on other freeways as well, may be due to differences in the driver population (i.e., the driver category, such as home-to-work, business, housewife) at different hours. In the morning peak, traffic is principally a "clean" buildup of home-to-work drivers intent on reaching their destination; but in the afternoon there is a

Figure 3.23. Daytime and nighttime speed distribution, Davison Expressway, Detroit, 1959. (Source: Ref. 21)

transition from more casual midday drivers to the work-to-home group.

A comparison of the distribution of day and night speeds is shown in Figure 3.23, based on data from the Davison Expressway in Detroit. The distribution in this case is for 1-min average speeds and shows the percent of minute intervals less than a given speed. The median speeds for daytime and nighttime are nearly identical, but the variation of nighttime speeds is much greater. Two-thirds of the daytime minute average speeds are within 3 mph of the median daytime speed, whereas less than one-half of the minute average speeds are within 3 mph of the median nighttime speed.

As noted in the foregoing examples, speed distributions and averages vary by hours of the day as the result of many influences. Driver characteristics, trip purposes, visibility, and volume-capacity effects are all related to daily speed variations.

Average Speed by Lanes

For any given time of day there is also a variation in average speed for each lane of multilane freeways. This effect is seen in Table 3.14, which presents mean speeds by lane compiled from several sources.

In every instance but one, the slowest average speed was found in the shoulder lane (lane 1). This is consistent with the prevalent pattern in the United States, where slow-moving vehicles keep to the right or shoulder lane.

Another factor influencing freeway lane speeds is the number of entrances and exits. The influence of adjacent ramps on lane average speed was demonstrated in a study of the Gulf Freeway which compared speeds before and after ramps leading to the freeway were closed to traffic (*10*). Based on 5-min observations, the volume in lane 1 adjacent to the ramps showed little, if any, increase when the ramps were closed. On the other hand, average speeds in lane 1 showed a substantial increase after the ramps were closed, in one case from 23 mph to 36 mph. This increase in speed was attributable to a decrease in turbulence in lane 1.

The values given in Table 3.14 indicate speed differentials between lanes under mod-
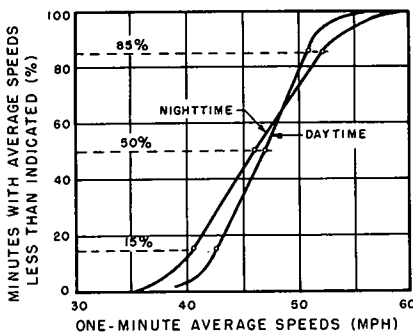
TABLE 3.14—MEAN SPEEDS BY LANES

LOCATION	SOURCE	MEAN SPEED (MPH)			AVG. RATE OF FLOW PER LANE
		LANE 1	LANE 2	LANE 3	
New Jersey Turnpike	Ref. 41	46	55	60	1,120
Eastshore Freeway, Calif.	Ref. 23	46	51	53	1,423
Pasadena Freeway, Calif.	Ref. 23	42	44	46	1,756
Gulf Freeway, Texas	Ref. 10	43	X	41	1,543
Ford Expressway, Mich.	Ref. 3	42	46	47	— [a]
Davison Expressway, Mich.	Ref. 3	45	48	47	— [a]
Santa Ana Freeway, Calif.	Ref. 23	44	45	None	1,963
North Sacramento Fwy., Calif.	Ref. 23	46	50	None	1,449
Merritt Parkway, Conn.	Ref. 32	47	51	None	1,095
Hutchinson River Parkway, N. Y.	Ref. 42	41	45	None	1,000

[a] 24-Hour average.

erate to heavy volumes. Generally, the average speed differential between lanes will show a wider range than that indicated in Table 3.14 under low volumes, with this differential narrowing as the volume increases.

Speed Distributions

Previous discussion has been concerned with the effect of various factors on average speeds. Except in unusual circumstances, however, individual vehicle speeds are distributed about the average. Figure 3.24 depicts typical distributions for uncongested level tangent sections of rural highways in 1941 and 1958, based on information supplied by a number of states, and for the New Jersey Turnpike and the Kansas Turnpike based on information supplied by their operating authorities.

Prior to 1941 the average speed for passenger cars was in the range between 42.5 and 48.5 mph, as shown in curves A and B, Figure 3.24. For higher speed highways in 1941 (curve B), the middle 70 percent of the drivers traveled within a range of 18 mph, from 39 to 57 mph. Average speed for all rural highways in 1958 had increased to 52.8 mph (curve B'), with the middle 70 percent of the drivers ranging from 44 to 61 mph, or a spread of 17 mph.

Curve C is a typical speed distribution for passenger cars on a rural freeway with a well-enforced 60-mph speed limit. The average speed is 55.5 mph and the middle 70 percent of the drivers were observed in the 16-mph range from 47 to 63 mph.

Curves D and D' show typical speed distributions for higher speed limits or light enforcement on high-type facilities. In each case, the average speed is 64.2 mph, and the middle 70 percent of drivers travel in a broader range between 55 mph and 73 to 75 mph.

The curves of Figure 3.24 are based on observations made when capacity limitations did not affect the drivers' choice of speed. As volume increases, drivers are less able to choose their own speed, the faster drivers being forced to decrease speeds more than the slow drivers, and the range of speeds is reduced.

A specific illustration of the influence of rate of flow on speed distributions is shown in Figure 3.25, derived from data on passenger car speeds in lane 2 of the John Lodge Expressway in Detroit (11). At a mean lane rate of flow of 954 vph, the median speed in lane 2 was 49 mph and the middle 70 percentile range was 9.6 mph. Increasing the rate of flow to 1,977 vph per lane lowered the median speed to 46 mph and the 70 percentile range to 8.8 mph. This relationship between volume and speed provides one method for capacity determinations as developed in this manual.
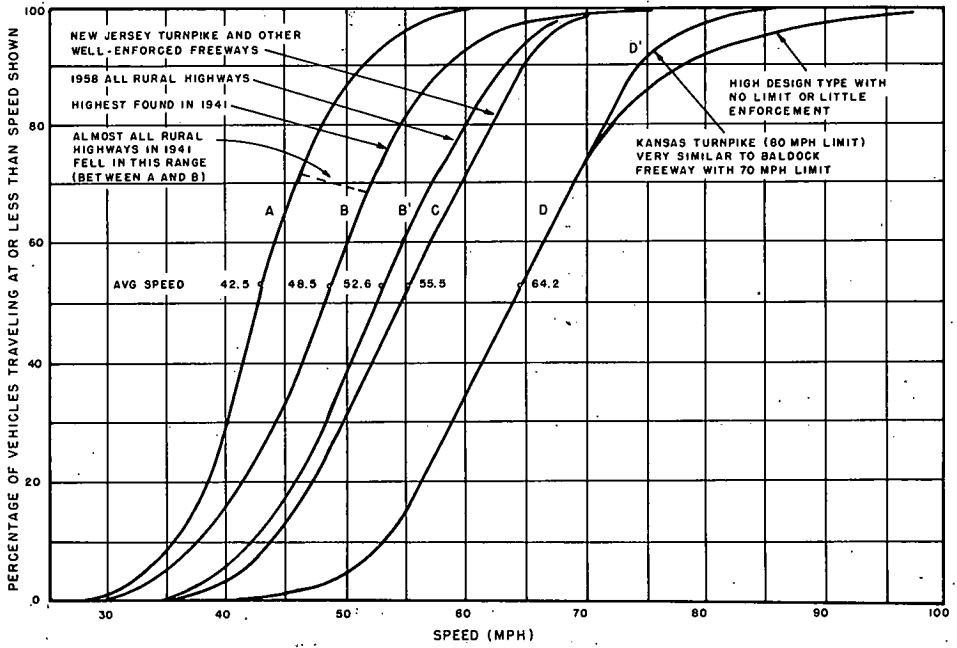
Figure 3.24. Distribution of normal passenger car speeds.
(Source: *Freeway Operations*, Inst. of Traffic Eng., 1960)
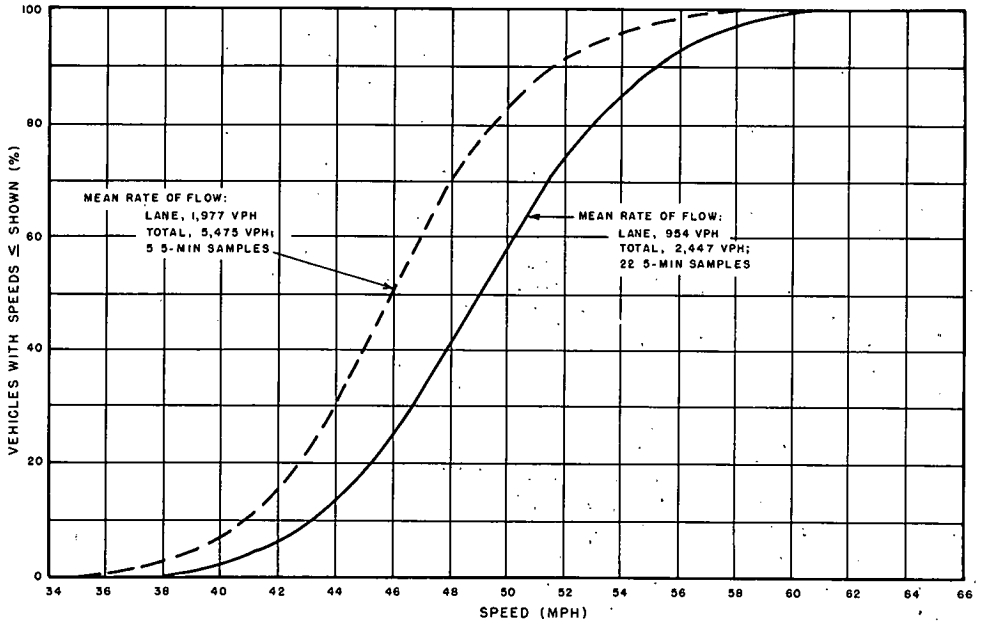
Figure 3.25. Passenger vehicle velocity distribution in northbound and southbound middle lanes,
Lodge Expressway, Detroit, 1957.
(Source: Ref. 11)

More general illustrations of this speed-volume relationship for certain broad classes of highway are presented in Figures 3.26, 3.27, and 3.28, which show for freeways, ordinary multilane highways, and two-lane highways, respectively, speed distributions likely to be found at each of several different approximate volume levels. These curves are generalizations for relatively ideal conditions, developed from a variety of recent speed distribution and average speed data on file at the Bureau of Public Roads. (The letters A through F on each chart refer to levels of service represented, as described in Chapter Four and used throughout the remainder of the manual).

SPACING AND HEADWAY CHARACTERISTICS

Capacity studies of intersections, weaving areas, ramps, and tunnels, and other analyses of roadway characteristics, have required the investigation of spacing and headway characteristics. Vehicular spacing also has application in predicting arrival rates at a point, testing the randomness of traffic flow, designing vehicular storage lanes, estimating gaps and delays at vehicular and pedestrian crossings, developing traffic control warrants, and timing traffic signals. The following discussion introduces vehicular spacing characteristics as an important determinant in traffic operations.

Mathematical Relationships

Spacing is the distance measure and headway is the time measure from head to head of successive vehicles. Thus, one mile of roadway includes spacings totaling one mile and one hour of traffic flow includes headways totaling one hour. Spacing and headway may be considered for each lane separately, for all lanes in one direction, or, in special cases, between all vehicles regardless of direction. These two measures thus describe the longitudinal arrangement of vehicles in a traffic stream.

The relationship between spacing and headway is dependent on speed, with

$$\text{Headway (sec)} = \frac{\text{Spacing (ft)}}{\text{Speed (ft/sec)}} \quad (3.1)$$

This equation is clearly true for pairs of
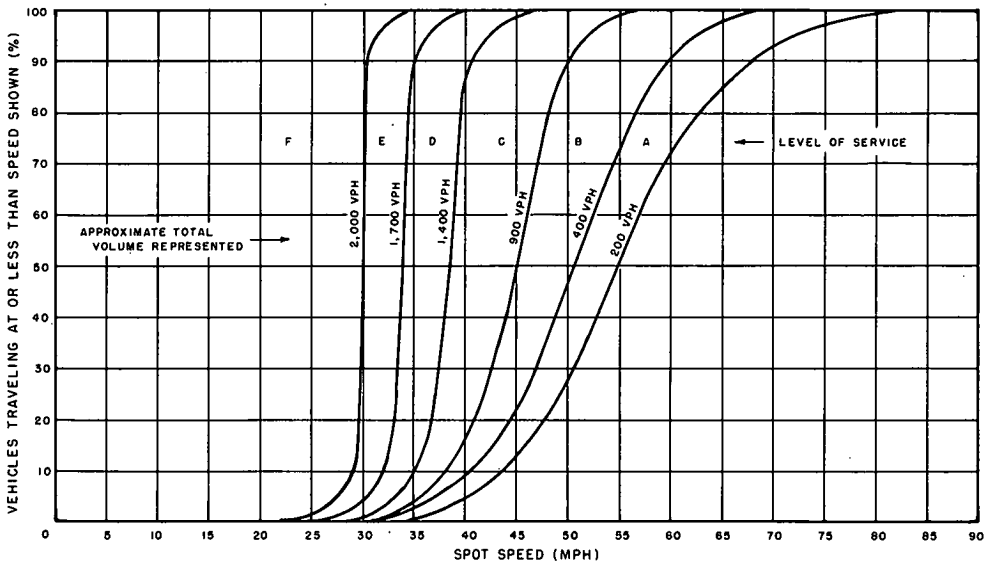
Figure 3.26. Typical distribution of passenger car speeds in one direction of travel under ideal uninterrupted flow conditions on freeways and expressways.
(Source: BPR, combined data from various studies)
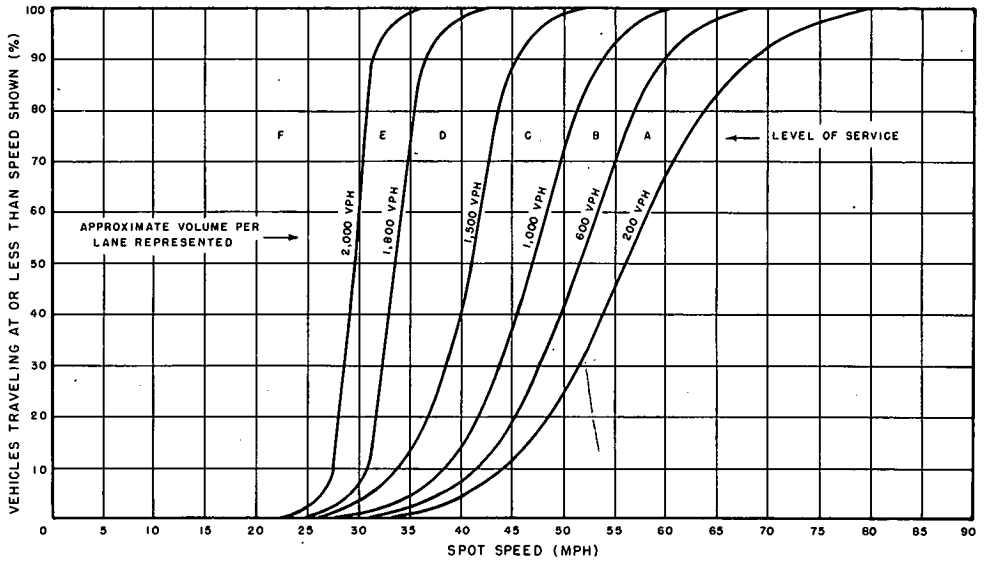
Figure 3.27. Typical distribution of passenger car speeds in one direction of travel under ideal
uninterrupted flow conditions on multilane rural highways.
(Source: BPR, combined data from various studies)
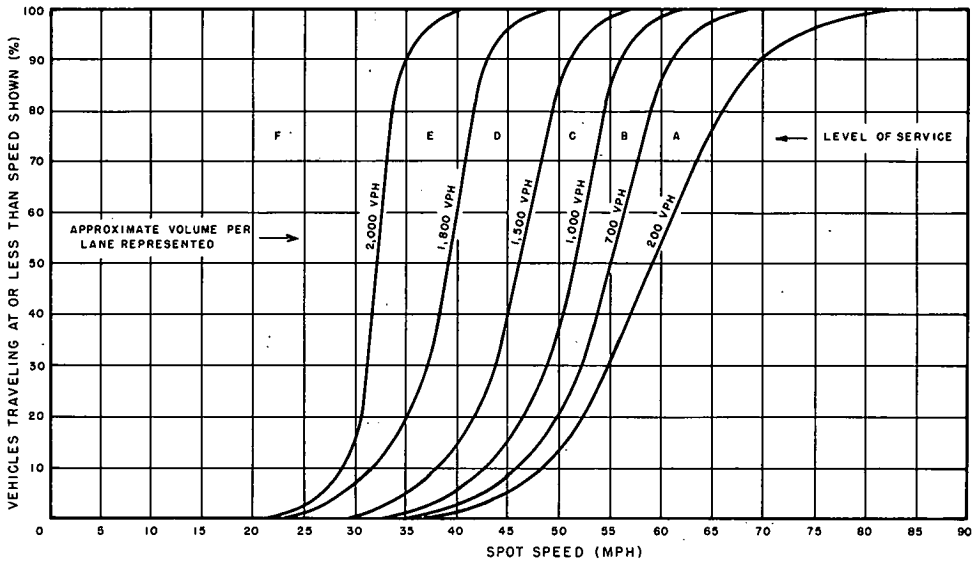
Figure 3.28. Typical distribution of passenger car speeds for both directions of travel under
ideal uninterrupted flow conditions on two-lane rural highways.
(Source: BPR, combined data from various studies)
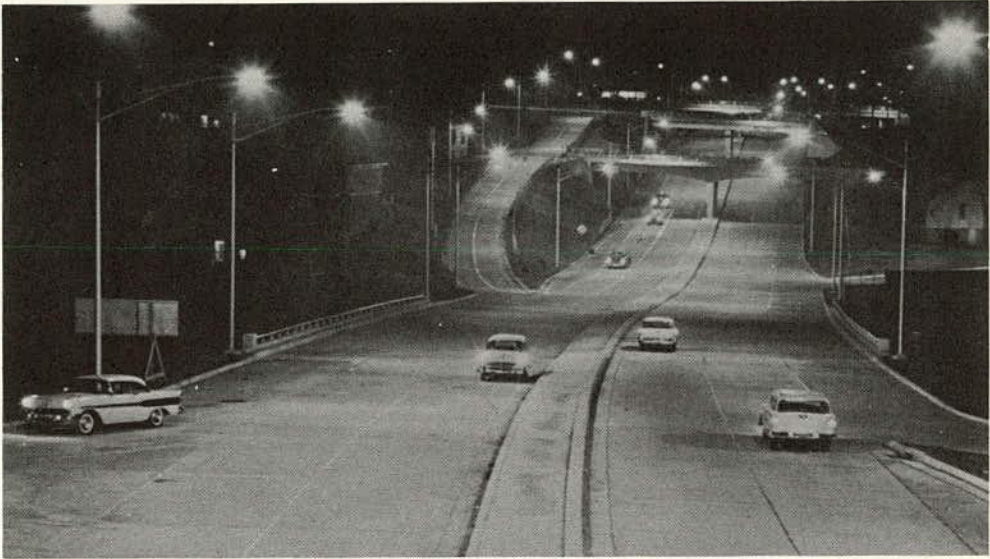
Urban freeway under low nighttime traffic flow.

vehicles or a traffic stream operating in a steady state, but becomes much more complex when individual vehicular speeds vary considerably.

A relationship between average spacing and density exists, as follows:

Density (veh/mile) =

$$\frac{5,280 \text{ (ft/mile)}}{\text{Avg. spacing (ft/veh)}} \qquad (3.2)$$

A similar relationship between average headway and volume may be expressed as

Volume (vph) =

$$\frac{3,600 \text{ (sec/hr)}}{\text{Average headway (sec/veh)}} \qquad (3.3)$$

Spacing as a Measure of Capacity

Although volume may be the most significant measure of traffic demand on a facility, spacing and headway affect the individual road user to a greater degree and are thus more directly related to the level of service. Spacing and headway give the driver traveling within the traffic stream his sense of freedom of movement or congestion and of relative safety, and continuously affect his choice of speed and position of his vehicle. His decisions in weaving, merging, passing, and car-following operations are predicated on his judgment of suitable gaps between vehicles. The frequency and length of gaps also govern his ability to enter or cross the traffic stream in question. Because spacing greatly affects the individual vehicle operation, the driver's reactions under various conditions have profound effects on highway capacity.

Much of the earliest work in highway capacity used assumed spacing between vehicles as the criterion. Fundamentally, the amount of traffic carried per unit of time varies directly with the speed and inversely with the spacing between vehicles. Therefore, considering a single lane of traffic for simplicity,

$$\text{Volume} = \frac{\text{Speed}}{\text{Spacing}} \qquad (3.4a)$$

or

Volume (vph) =

$$\frac{5,280 \text{ (ft/mile)} \times \text{Speed (mph)}}{\text{Spacing (ft/veh)}} \qquad (3.4b)$$

Using this reasoning, many early research-ers and authors determined the maximum capacity of a traffic lane by assuming certain minimum spacings at various speeds. In some of their studies, minimum spacings were computed by use of such factors as driver reaction time, braking distances, and coefficients of friction. In others, minimum spacing as a function of speed was derived from field observations or photographic stud-ies of vehicles traveling in queues so that each one could be assumed to be traveling at minimum spacing.

Much of this earlier work was summarized in the original (1950) edition of this manual. Some of these results were remarkably close to the speed-spacing relationships found in more recent studies, especially for the lower ranges of speed. The main assumption made in most cases was that for maximum flow all or nearly all of the vehicles must be traveling at minimum spacing.

Other studies have found that, generally, drivers perform by using the criterion of potential time to a collision point, with average minimum headway a constant, re-gardless of speed. Minimum headways vary from $\frac{1}{2}$ sec to 2 sec, depending on the driver and traffic conditions, with an average of about $1\frac{1}{2}$ sec (12, 13). This value corre-sponds to a rate of flow per lane of 2,400 vph, which has been observed for short periods under ideal environmental condi-tions on certain lanes of various freeways, usually the median lane. This rate, however, occurs under too specialized conditions to be considered a criterion for capacity.

Headway Distribution and Random Flow

If all vehicles using a highway were equally spaced, determination of maximum volumes or levels of congestion would be a simple matter. However, vehicles do not move at uniform headways; rather, they tend to form groups, even at low volumes. For each level of traffic volume there will be an average headway. However, individual headways will show a large variation, with many vehicles queuing at short headways and others separated by relatively large time gaps.

Figures 3.29 and 3.30 show headway dis-tributions for vehicles traveling in the same direction on typical two-lane and four-lane rural highways for various volumes during uninterrupted flow. Under nearly all vol-ume conditions, approximately two-thirds of the vehicles are spaced at, or less than, the mean headway between vehicles. In Figure
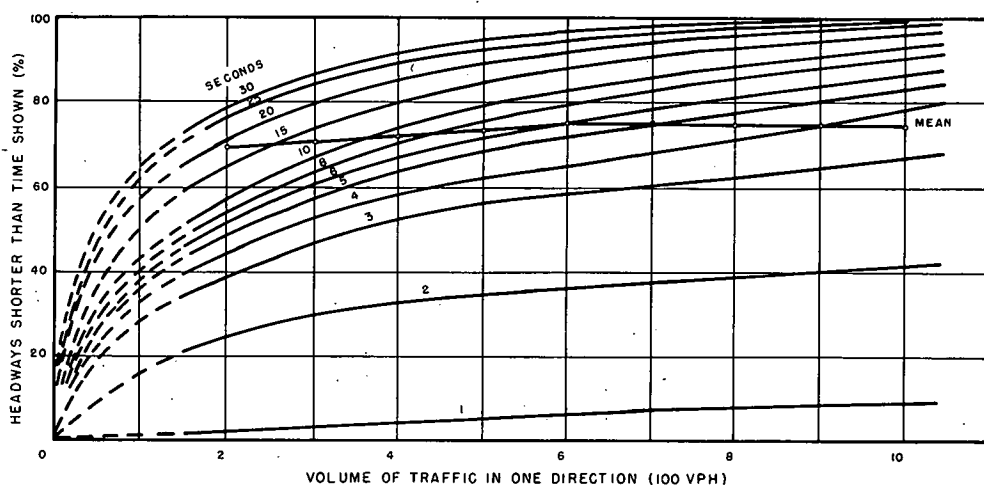
Figure 3.29. Frequency distribution of headways between successive vehicles traveling in the same direction at various traffic volumes on typical two-lane rural highway.
(Source: Ref. 43)
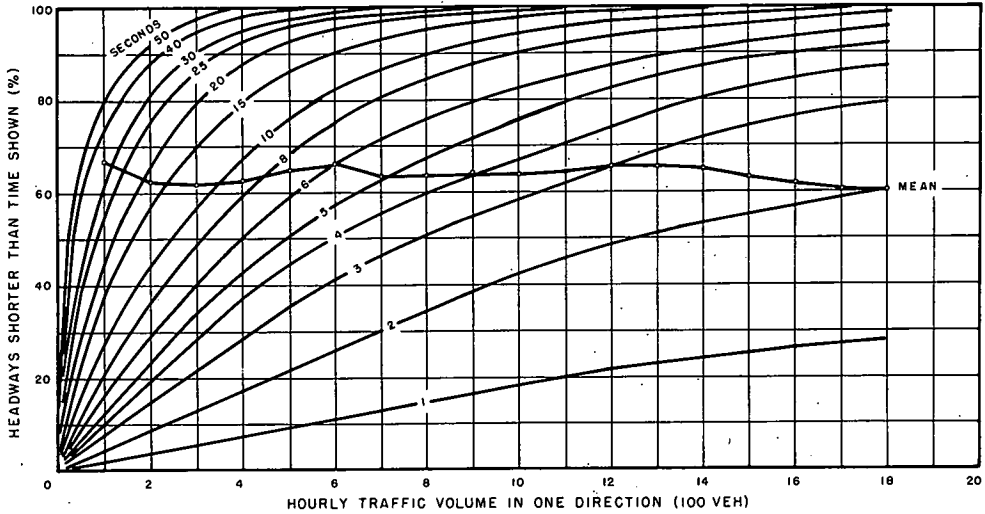
Figure 3.30. Frequency distribution of headways between successive vehicles traveling in the same direction at various traffic volumes on typical four-lane rural highway.
(Source: Ref. *43*)

3.30, for example, it can be seen that at an hourly volume of 600 vehicles in one direction (or a mean headway of 6 sec), approximately 400 vehicles will be 6 sec or less behind the car ahead.

A study of Michigan freeways presented headway data in a somewhat different manner (*3*). Figure 3.31 shows the headway distributions related to 1-min flows separately for each lane of the Edsel Ford Expressway. The heavy curve in each case indicates the mean headway at various 1-min flow levels; the lighter curves represent the 15, 50, 85, and 100 percentile levels for each lane. The 1-min flows per lane were classified into seven groups by flow rate (6-10, 11-15, 16-20, 21-25, 26-30, 31-35, and 36-40 vpm) and the distribution of headways for each group is presented as a crosshatched area. In lane 3 the mode is between 0.6 and 1.0 sec, whereas for lane 1 the mode ranges from 1.5 to 2.0 sec. As 1-min flows increase, the distribution of headways is more peaked and the mode between 0.5 and 2.0 sec becomes more pronounced.

When these data were combined with data from other Michigan multilane high-

ways (*3*), it was found that in all cases the mode (most frequently occurring value) was less than the median (50 percentile level) and the median was less than the average headway. Approximately two-thirds (64 to 69 percent) of the headways were less than the mean headway, in agreement with the data of Figure 3.30.

Further description of vehicular spacing characteristics can best be made in mathematical terms. Under certain conditions vehicular spacing or vehicle arrival rates at a point follow a random distribution; that is, the position of each vehicle is independent of any other vehicle and equal segments of the road are equally likely to contain the same number of vehicles. Such a distribution is given by the Poisson distribution

$$P(x) = \frac{e^{-m}m^x}{x!} \qquad (3.5)$$

in which

$P(x) =$ probability of exactly x occurrences;
$\quad x =$ number of occurrences;
$\quad e =$ base of Napierian (natural) logarithms ($= 2.7183$); and
$\quad m =$ average expectation of occurrence.
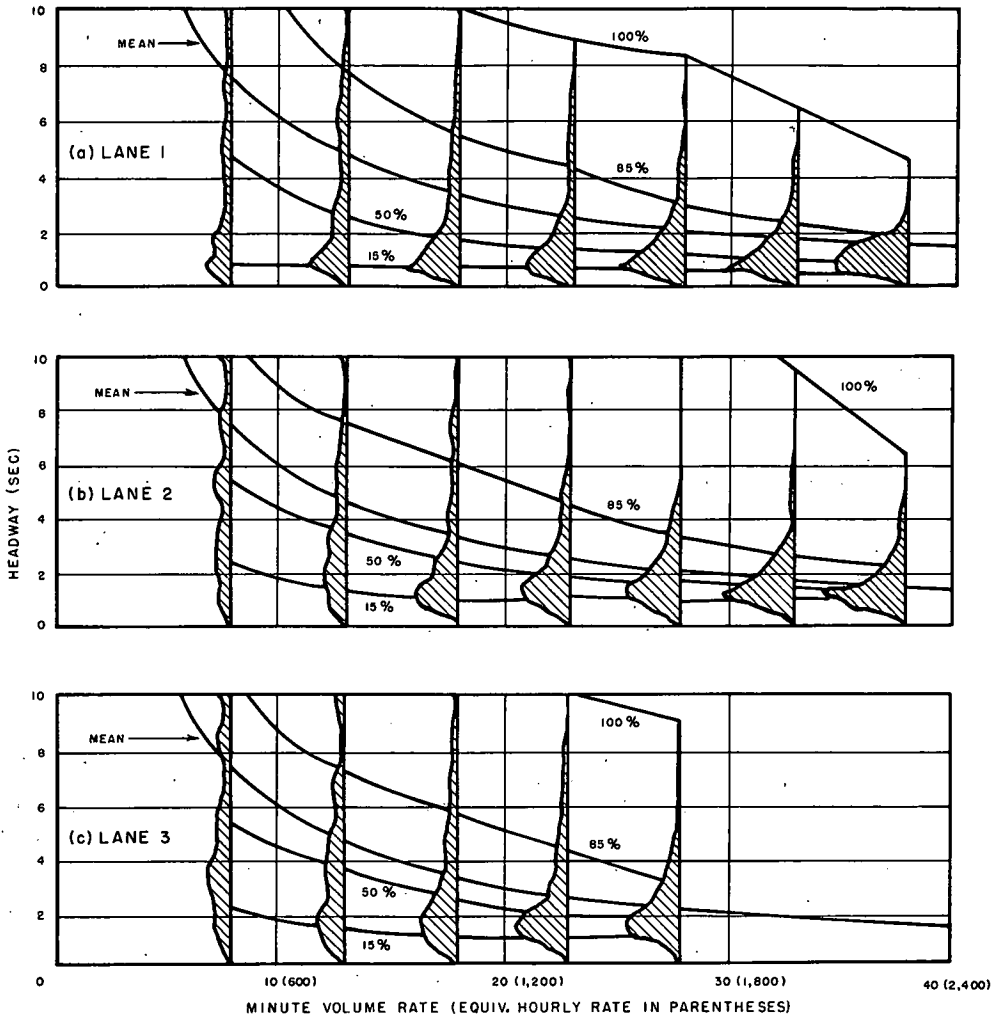
Figure 3.31. Lane headway distribution related to traffic flow, Ford Expressway, Detroit, 1957.
(Source: Ref. 21)

The Poisson distribution is most useful in dealing with the distribution of discrete events, such as the arrival of vehicles within a given time interval. The distribution of headways between vehicles is a continuous variable and is exponential in nature. This exponential distribution, derived from Poisson for the condition that no vehicles arrive during a given time interval, is given by

$$P(h \geq t) = e^{-qt} \qquad (3.6)$$

in which

$P(h \geq t)$ = probability of a time gap equal to or greater than t;
h = headway, in sec;
t = time, in sec; and
q = flow per second.

The solid line in Figure 3.32 is the computed exponential distribution, but the points shown are obtained from Figures 3.29 and 3.30 for two-lane and four-lane rural highways carrying 500 vph. At this volume, the Poisson

distribution provides a good fit for four-lane highways, but not for two-lane highways.

Although the Poisson distribution may approximate the distribution of headways in a traffic stream, two factors limit its direct application. First, the theoretical curve distributes headways continuously over the entire range of headways; yet, obviously, in practice there exists a minimum headway that cannot be diminished. Except for multilane highways, where two vehicles in different lanes in the same direction of travel can maintain a lesser headway, no spacings exist at values less than perhaps 0.5 sec or 30 ft, inasmuch as each gap must also include one vehicle. The poor fit of the data for two-lane highways in Figure 3.32 demonstrates another effect, the concentration of more vehicles into the short headway classes due to platooning. As volumes increase, more and more vehicles adopt short headways as they overtake, but cannot pass, slower moving vehicles. This bunching effect is prevalent on two-lane highways, but may also become a significant factor on multilane highways at heavy volumes.

Various authors have proposed modifications of the basic Poisson distribution to correct for these factors. One proposal is a translation of the exponential curve a small distance away from the origin to eliminate less-than-minimum headways (14). For the other factor, the platooning or "bunching" of traffic, a composite exponential distribution has been proposed $(15, 16)$. It has been hypothesized that a traffic stream is composed of a combination of free-flowing and restrained vehicles, each conforming to a Poisson distribution.

The equation for this composite distribution (16) is

$$p(h \geq t) = (1-\alpha)e^{-(t-\lambda)/(T_1-\lambda)} + \alpha e^{-(t-\tau/T_2-\tau)}$$

$$(3.7)$$

in which

$p(h \geq t) =$ probability of a headway, h, greater than or equal to the time, t;

$\alpha =$ proportion of the traffic stream in the restrained group;

$(1-\alpha) =$ proportion of the traffic stream in the free-flowing group;
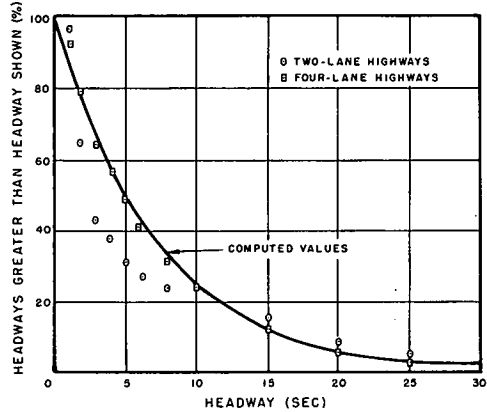
Figure 3.32. Computed and observed headways for typical two-lane and four-lane highways carrying 500 vph in one direction.
(Source: Data from Figs. 3.29 and 3.30)

$T_1 =$ average headway of free-moving vehicles;

$T_2 =$ average headway of restrained vehicles;

$\lambda =$ minimum headway of free-moving vehicles;

$\tau =$ minimum headway of restrained vehicles; and

$e =$ base of Naperian (natural) logarithms $(=2.7183)$.

This composite distribution has been used in recent studies of traffic flow on two-lane urban streets $(16, 17)$. Figure 3.33 shows a cumulative composite distribution fitted to experimental data from 585 samples of traffic flow on a two-lane urban street with rates of flow ranging from 150 to 1,200 vph (17). The experimental data are shown as circled points; the composite curve based on parameters calculated from observed data is represented by the solid line. It is important to note that the restrained vehicle is only significant in that portion of the curve representing headways of less than 6 sec.

The separation of flows into free-flowing and restrained vehicles has also been indi-
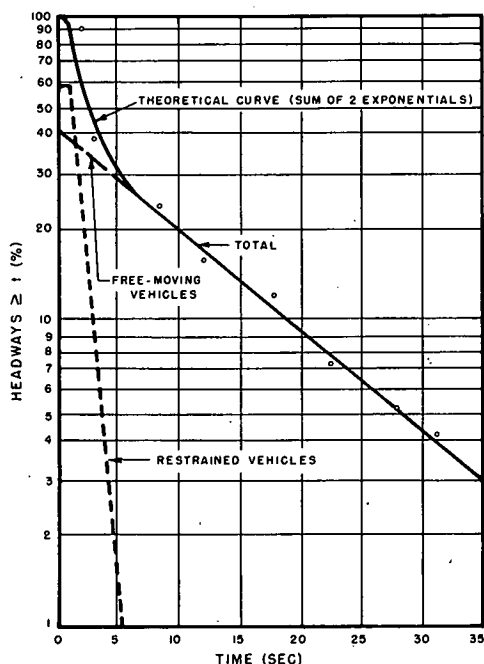
Figure 3.33. Computed and observed headways on a two-lane urban street.
(Source: Ref. *16*)

As a practical matter, warrants for design criteria and traffic control measures should be based on how the highway will function under different traffic flows. When a driver desires to cross a traffic stream from a stop condition, he will cross when a gap in the main traffic stream seems adequate to him. From studies on a rural four-lane highway in California, charts were prepared showing the probability of waiting a certain time for different-sized time intervals between arrivals of cars in either direction in the total two-way traffic stream. Figure 3.34 shows two of these charts giving, for various intervals, the probable waiting time which would not be exceeded 95 percent and 50 percent of the time at various volume levels.

Figure 3.35 shows the percentage of time occupied for all spaces equal to or greater than selected values and the percentage of time that spaces are in excess of certain values at various volumes, based on observations on typical two-lane and four-lane rural highways. These data, though recorded in the early 1940's, have proven to remain consistently valid and are included as evidence of the consistency of certain traffic characteristics.

cated in other studies of spacing characteristics. Car-following experiments have indicated that the correlation of behavior between successive vehicles decreases rapidly at a distance greater than 200 ft, with little or no correlation beyond 500 ft (*18, 19*). Earlier studies discussed in the original 1950 edition of this manual found no effect on the following vehicles when headways exceeded 9 sec.

The distribution of headways derived from either a basic or modified Poisson distribution has many applications in traffic investigations. One would be the comparison of observed with theoretical headway distributions for various volume levels. Either substantial deviation from a random distribution or a large percentage of vehicles traveling within the restrained headways would give an index of the congestion being experienced by the traffic stream.

Another application is in estimating the number and length of gaps in a traffic stream at pedestrian or vehicular ingress points.

Effects of Traffic Interruptions on Headways

Obviously, the frequency of time spacings will be materially different from random flow at or downstream from flow interruptions, such as traffic signals. A traffic signal has the effect of platooning all vehicles and vehicles leave such an interruption under constrained conditions. As the platoon moves down the highway, it tends to spread over both time and distance. If succeeding interruptions are not present, at some distance downstream headways again become random.

Figure 3.36 shows the frequency distributions of vehicle arrivals at several points downstream from a traffic signal on a four-lane divided urban facility in California. Although there were several minor intersections in this stretch of highway, traffic was assumed to be uninterrupted downstream from the traffic signal within the limits of study. As distance from the signal increased,

the platooning effect decayed as the faster and slower vehicles detached themselves from the group. A tendency for vehicles to change lanes with increasing distance downstream was also noticed. In another study in Michigan (20) it was found that the spread of a platoon of vehicles could be described by a simple kinematic model. The model assumes that cars in a platoon travel at fixed speeds which are normally distrib-

uted about some mean speed. Experimental results confirm that the model accurately describes the spreading of a platoon in medium traffic moving without interference.

Knowledge of the effects of traffic interruptions on headways is necessary in evaluating many traffic engineering measures. On one hand, the presence of traffic signals upstream will affect the distribution of acceptable gaps for vehicles or pedestrians
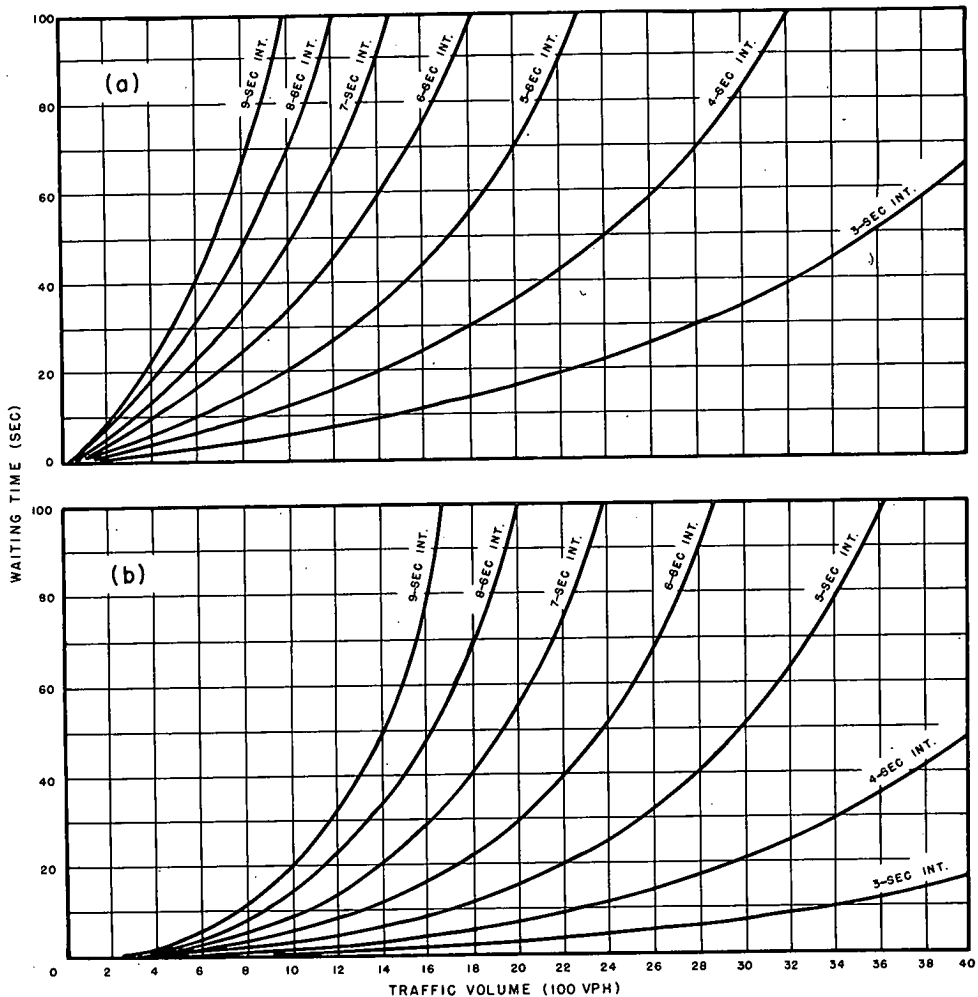
Figure 3.34. *Waiting time for selected intervals at various volumes with (a) probability of 95 percent and (b) probability of 50 percent.*
(Source: Ref. 44)
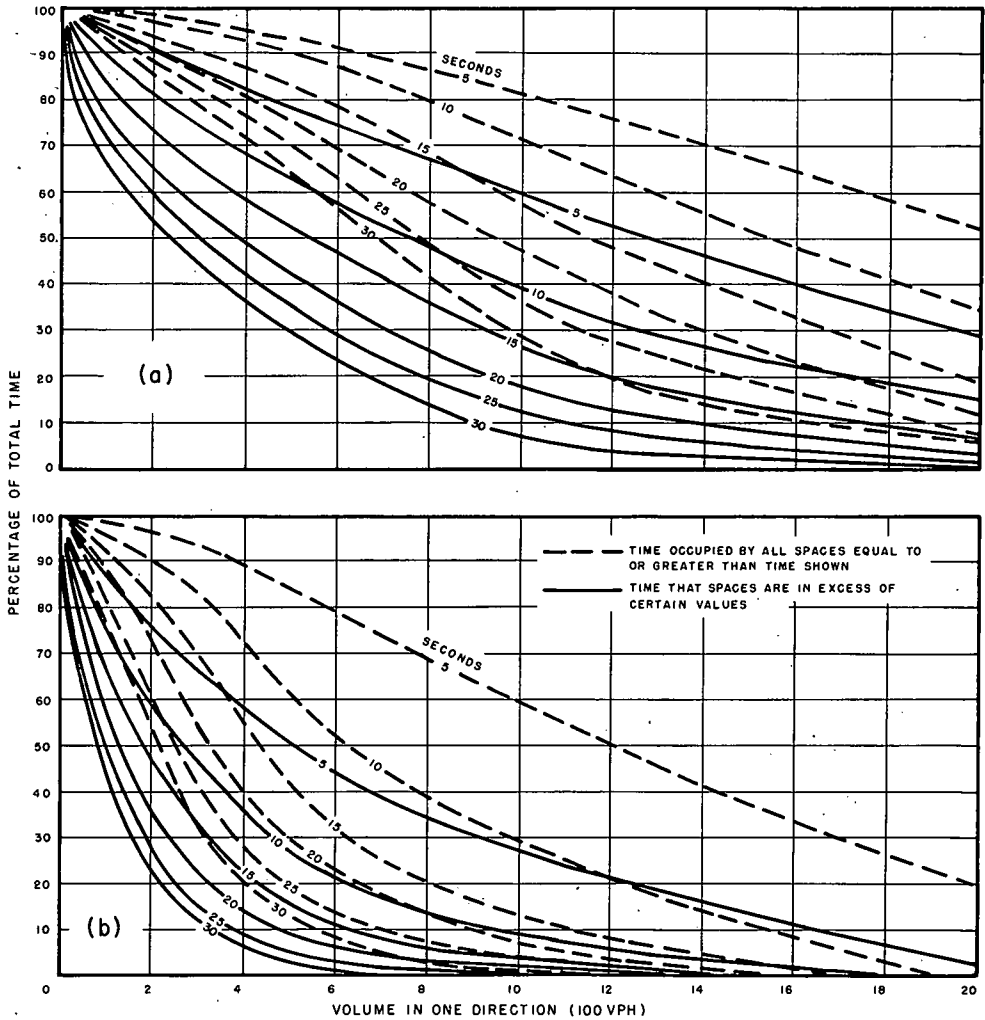
Figure 3.35. Percentage of total time occupied by various time spacings between vehicles travel-ing in the same direction and percentage of total time that spacings are in excess of certain values on (a) typical two-lane rural highways and (b) typical four-lane rural highways.
(Source: Ref. 45)

wishing to enter or cross the traffic stream. On the other hand, retention of platoons is desirable in progressive signal systems. The locations and conditions of application are too varied to permit presentation of specific criteria, inasmuch as vehicles entering or leaving the traffic stream within a section, or operating erratically within it, create variances from a normal pattern.

Density as a Measure of Conditions

As previously stated, spacing can be alternatively expressed in terms of density:

Density (veh/unit length of roadway) =

$$\frac{1}{\text{Avg. spacing (length of roadway/veh)}}$$

(3.8)

Considering the units in which it is expressed (vehicles per given distance), it can be seen that density describes the conditions along a *length* of roadway, rather than at one given point. Headways, on the other hand, better describe point conditions.

As defined in Chapter Two, space mean speed is the average speed of all vehicles on a given length of roadway at an instant in time. Density describes the number of vehicles on a given length of roadway, also for an instant in time. If both are expressed in comparable units (density in vehicles per mile and space mean speed in miles per hour), their product is a rate of flow. Therefore, a basic relationship exists, with

Rate of flow (vph) =

$$\text{Space mean speed (mph)} \times \text{Density (veh/mile)} \qquad (3.9)$$

In this sense, the derived flow is the rate for the instant in time being studied, although it may be expressed in vehicles per hour.

The relationships between speed, flow, and density are explored further in the succeeding section of this chapter; each is a significant indicator of operating conditions and may require consideration in developing capacity criteria.

RELATIONSHIPS OF SPEED, FLOW, AND DENSITY

This section enlarges upon and summarizes the previous discussion of speed, flow, and spacing. Pure relationships are investigated where possible, such as the effect of flow on speed with all other variables held constant. Although difficult, numerous studies have been successful in approximating this laboratory approach.

The principles of physics, dynamics, hydraulics, and the laws of various sciences are being applied to traffic research with increasing success. With computers, traffic flow is being simulated with an increasing degree of realism and these simulations are serving as the basis for certain studies.
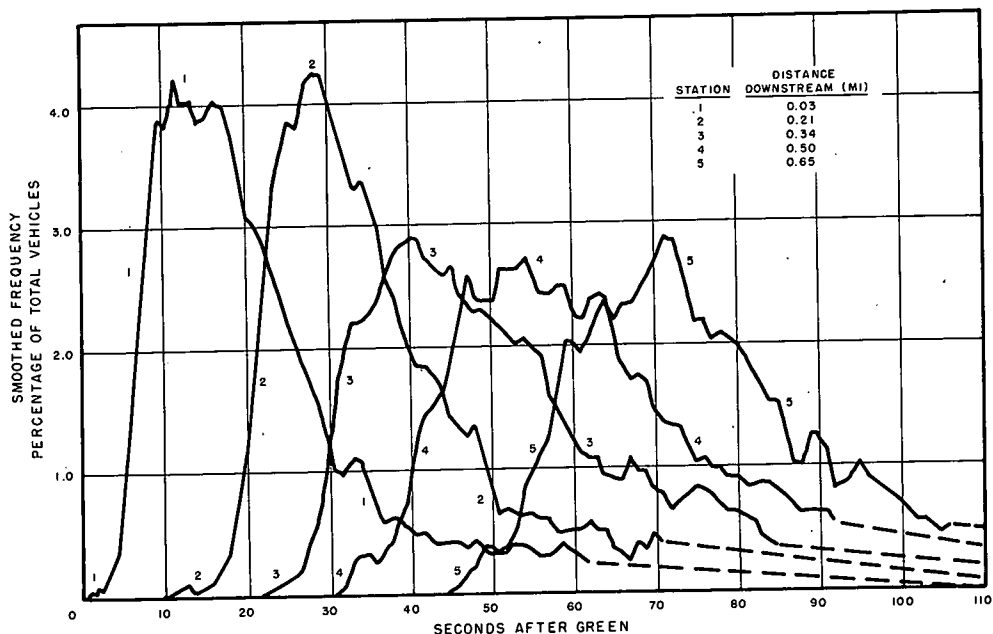
Figure 3.36. *Frequency distributions of vehicle arrival times at five stations downstream from a traffic signal, California.*
(Source: Ref. 46)

Based on probability distributions, the aggregate behavior of drivers can be predicted under various given conditions. However, the traffic stream is not homogeneous with regard to either drivers or influence of environmental factors, and cannot, therefore, be precisely reproduced. As a result, few practical applications to specific situations have been possible as yet.

Nevertheless, a theoretical approach is highly useful, with or without computer application, because it is indicative of what the traffic observer might expect to find in the field under the controlled, or ideal, circumstances seldom if ever actually found in practice. For instance, it is becoming increasingly evident that few, if any, researchers have recorded truly "ideal" through flows on urban freeways. Practically all such urban data examined in the course of preparation of this manual had to be qualified in some way, to recognize the presence of nearby on- and off-ramps, changes in number of lanes, "tunnel effects" under bridges, or the like, which influenced the flow, probably adversely. Given theoretically established limits, the engineer can better visualize the true magnitude and nature of his specific operational problems.

Augmenting theory, a thorough knowledge of what has already been found in carefully conducted and analyzed field studies is indispensable to a complete understanding of the relationships among vehicle speed, flow, and density. This manual brings together in one place the experience gained from many studies throughout the United States. With this experience as a starting point, the individual researcher can augment his own direct observations of local conditions and be helped to sound conclusions.

A combination of theoretical and field studies is thus considered to be the best overall approach. Accordingly, the following paragraphs are arranged first to provide a theoretical expectation about a particular relationship, and second to report selected field studies that tend to support the theory. References are given for further study of details. Symbolic notation is occasionally used to conserve space. Speed-flow, speed-density, and flow-density relationships are considered in that order.

Speed-Flow Relationships

This section presents only the fundamentals of speed-flow and speed-volume relationships. Because many of the capacity and level of service criteria presented in the remainder of this manual are based on these relationships, they will be discussed in more specific detail where appropriate in the chapters that follow.

UNINTERRUPTED FLOW

The fundamental speed-flow relationship for a given population of drivers can be simply stated as follows: As traffic flow increases, the space mean speed of traffic decreases. This relationship holds true throughout the range of free flow and impending congestion, up to the point of critical density, or the density at maximum flow. At and beyond this point, however, it no longer applies; both rates of flow and space mean speed then decrease with an increase in density. This relationship applies to a roadway section, rather than to a point. Although studies of very short sections may produce erratic results, longer sections typically produce reasonably smooth and measurable relationships.

Space mean speed, as referred to in the foregoing, is an average and does not indicate the maximum attainable speed at a given flow. There is, of course, some range of speeds at any given flow, as the preceding section has shown. At low flows the range of individual vehicle speeds may be great; at higher flows the range narrows. Best-fit curves, connecting the space mean speeds, generally are used to represent the speed-flow relationship for uninterrupted flow, but boundary curves connecting the maximum speeds for given flows also are used on occasion.

There are varying degrees of uninterrupted, or continuous, flow. Although it may be more or less congested, uninterrupted flow means an absence of traffic signals, stop signs, or other traffic control interruptions. At one extreme the movement of vehicles may be very irregular due to mar-

ginal frictions, such as strip commercial developments. At the other extreme the movement of vehicles may be quite smooth in the absence of such frictions. This suggests that different roadways have different speed-flow curves. For example, the freeway driver expects minimum marginal interferences and maintains shorter headways. This may explain why speed of vehicles on a freeway should decrease less rapidly with increasing flow than that on a highway with no access control.

There are a number of other factors which affect the speed-flow relationship, including the "character" of traffic, the weather, the accident record, and other difficult-to-assess factors. They are important because when they are not explicitly recognized and dealt with they may obscure the real speed-flow relationship. The various studies cited in this section reportedly included no such unusual factors which would affect results.

Investigations conducted on an extensive scale have shown that a straight line reasonably represents the speed-flow relationship in the range below critical density, for uninterrupted flow conditions on all ordinary multilane highways without access control, as well as on most four-lane freeways.

These investigations also indicate that speed-flow relationships for freeways of more than four lanes are somewhat curved, reflect-

ing faster speeds at intermediate flow rates than are found on other highways. Thus, they appear flatter than those for other highways.

Results of a study on the Ford Expressway in Detroit, Mich. (21), are shown by the curve labeled Detroit in Figure 3.37. In this study the minute rate of flow and average speed for all the minutes in a day are arrayed and plotted. The curve shown is for the median lane. The curve labeled Chicago in Figure 3.37 represents aggregate hourly volume and speed data for all lanes from two separate sections of the Eisenhower (Congress St.) Expressway, and for one lane each of the Edens and Calumet Expressways and South Lake Shore Drive (22). For the 116 observations represented by this curve the correlation coefficient is 0.876. The curve labeled Los Angeles is based on 5-min volume and speed observations for all lanes in one direction on the six-lane Santa Ana Freeway during two evening peak periods (23).

Theoretically, under any uninterrupted-flow conditions, a rigidly-enforced speed limit lower than the average highway speed would result in a flatter average speed-flow curve than otherwise would be the case. The flattening here would result from lowering of the low-volume end of the curve, where normal average speeds cannot be attained due to the speed limit; at some
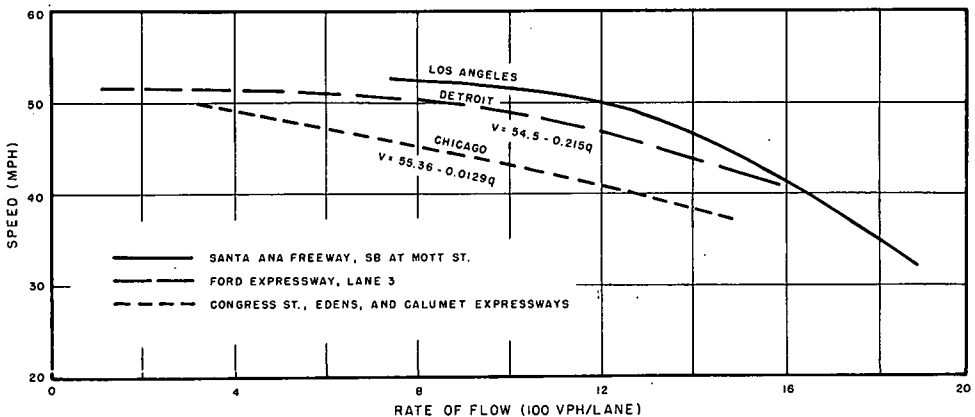
Figure 3.37. Speed-flow relationships for three different highways.
(Sources: Refs. 21, 22, 23)

Basic freeway
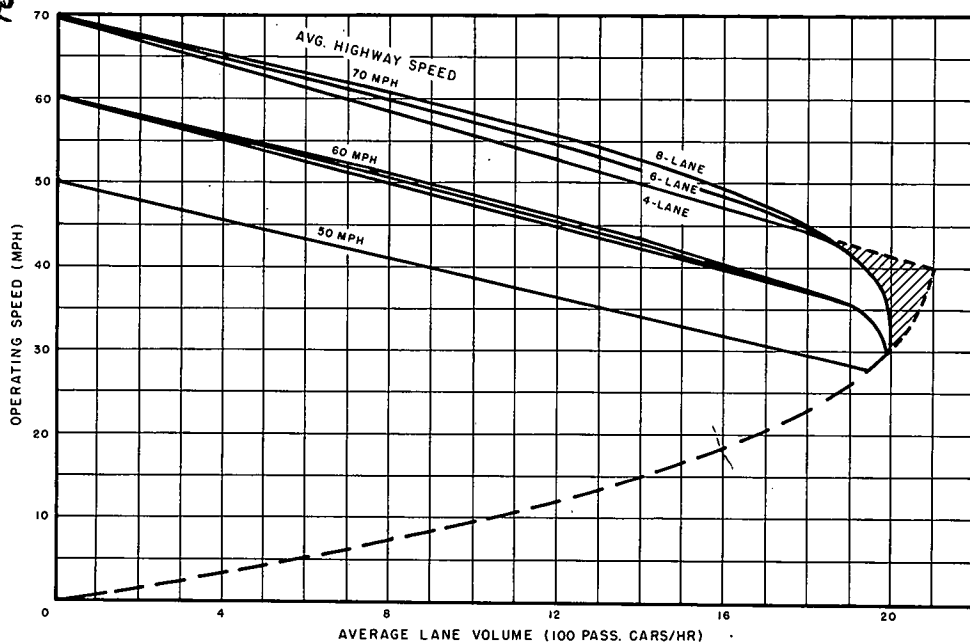
Figure 3.38. Typical relationships between volume per lane and operating speed in one direction of travel under ideal uninterrupted flow conditions on freeways and expressways.
(Source: BPR, combined data from various studies)

point, as volumes increased, the speed limit would no longer govern. This is in contrast to the freeway case previously described, where the flattening results from a raising of the high-volume end to reflect the higher speeds attained during heavy volume conditions, as compared to speeds on ordinary highways. Where these two effects both are found, as on heavily-used freeways with enforced speed limits, there may appear to be little change in average speed over a wide range of flow rates.

On two-lane highways, speed-flow relationships appear to take a somewhat wavy form which, however, does not depart greatly from a straight line in most instances (24). Trends toward higher speeds over the years have gradually raised the speeds typically found at the higher flow levels.

Throughout the procedural portions of this manual applying to uninterrupted flow, operating speed is used as a primary measure of level of service. Figures 3.38, 3.39, and 3.40 show typical relationships between

operating speed and volume, given ideal conditions, on freeways; ordinary multi-lane highways, and two-lane highways, respectively. Curves for less-than-ideal average highway speeds are also shown. These charts, like the speed distributions presented earlier, were developed from data on file at the Bureau of Public Roads. They are presented as illustrations only, and should not be used for actual problem solutions because they do not incorporate adjustments for the various adverse influences usually found on actual roadways.

The upper portion of each curve shows the relationship up to the point of critical density. Beyond this point, however, a further increase in flow causes the speed to decrease rapidly, with a marked simultaneous decrease in rate of flow. For example, in Figure 3.39, at a rate of flow of 1,400 vph per lane on an ordinary multi-lane rural highway, the space mean speed might range from 45 mph with free-flowing

These apply Two lane highways
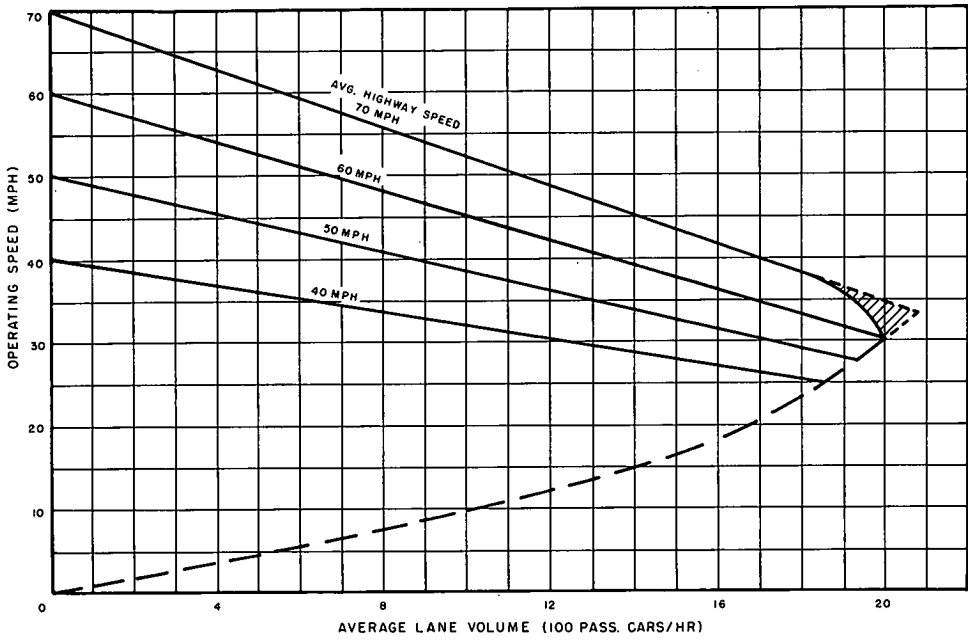
Figure 3.39. Typical relationships between volume per lane and operating speed in one direction of travel under ideal uninterrupted flow conditions on multilane rural highways.
(Source: BPR, combined data from various studies)
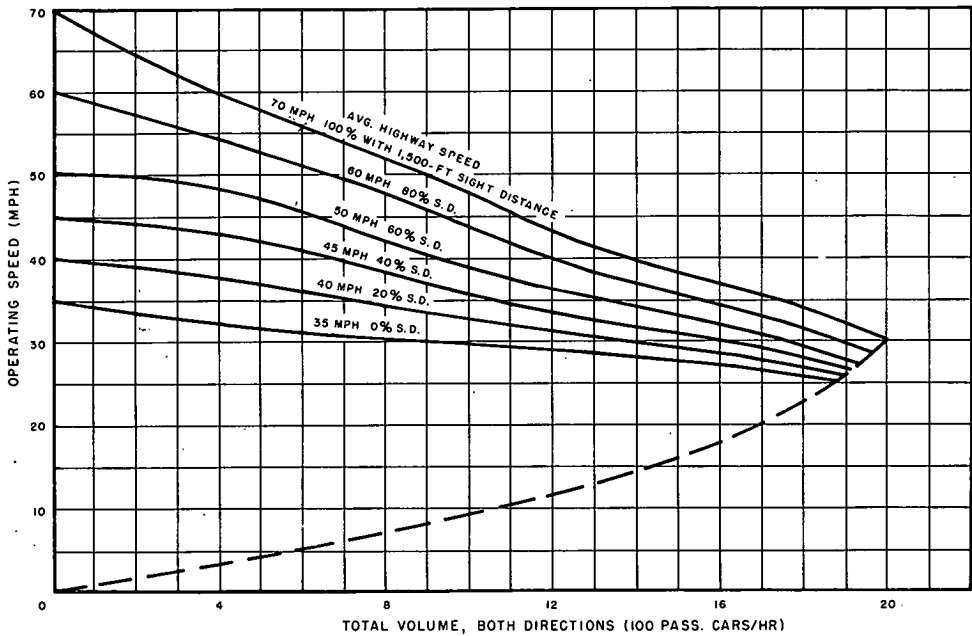
Figure 3.40. Typical relationships between total volume for both directions of travel and operating speed under ideal uninterrupted flow conditions on two-lane rural highways.
(Source: BPR, combined data from various studies)
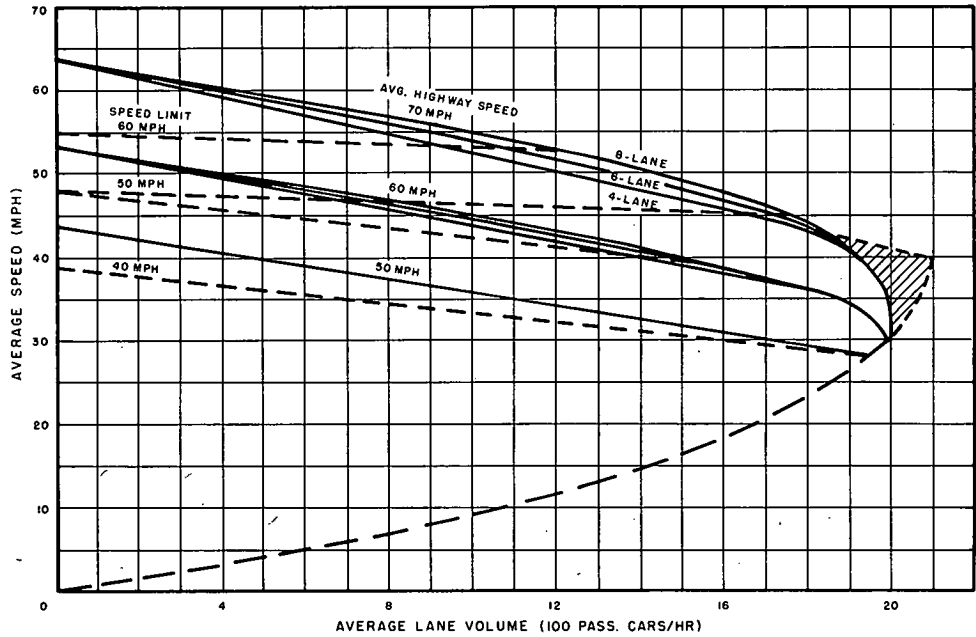
Figure 3.41. Typical relationships between volume per lane and average speed in one direction
of travel under ideal uninterrupted flow conditions on freeways and expressways.
(Source: BPR, combined data from various studies)
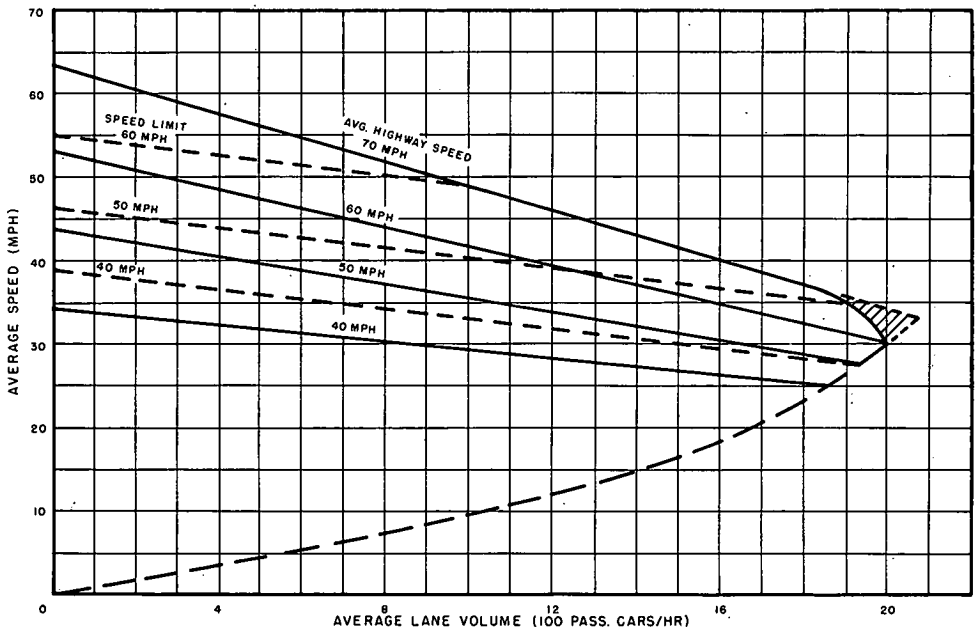
Figure 3.42. Typical relationships between volume per lane and average speed in one direction
of travel under ideal uninterrupted flow conditions on multilane rural highways.
(Source: BPR, combined data from various studies)
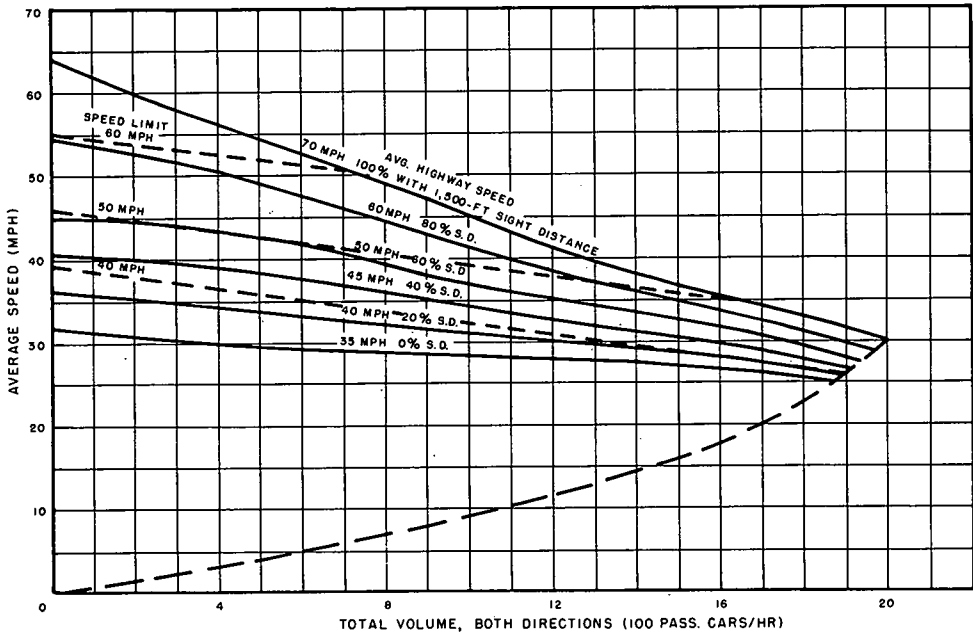
Figure 3.43. Typical relationships between total volume for both directions of travel and average speed under ideal uninterrupted flow conditions on two-lane rural highways.
(Source: BPR, combined data from various studies)

conditions to only 15 mph under highly congested stop-and-go conditions.

These charts portray actual hourly volumes, averaged across all traffic lanes, on highways having a high peak-hour factor (that is, constant high traffic demand throughout the hour). The shaded area at the right end of each chart represents highly unstable conditions. On freeways such volumes are occasionally found in one or two lanes (usually those nearest the median) but average hourly volumes of this magnitude over all·lanes are recorded too rarely to be considered as reasonably attainable.

If, instead, it were assumed that the charts represent flows (over short periods) rather than volumes (for full hour), short periods of operation in or even to the right of the shaded area might be expected.

Figures 3.41, 3.42, and 3.43 are equivalent charts showing the average speed-volume relationship, while Figure 3.44 presents examples of source material for freeways. On these several charts, the influence of en-forced speed limits is also shown. It will be noted that the average speed is always somewhat less than the operating· speed, at any volume level below capacity. These charts are included primarily to make clear the distinction between average speed and operating speed. Again, these charts are presented as illustrations, not as bases for computations.

INTERRUPTED FLOW

The speed-flow relationship is difficult to isolate under interrupted flow conditions. In the most common example, the city street with signalized intersections, both demand and capacity often are different on immediately adjacent segments. Also, maximum speed is frequently determined by external influences, such as signal progression timing and speed limits, rather than by driver desires. Thus, most studies of interrupted flow· characteristics have dealt with relatively short segments and have treated the relationship indirectly in terms

SOURCES

LOCATION	SYMBOL	TIME	NO. OF LANES	SPEED LIM MAX	SPEED LIM MIN	REPORTER
NEW JERSEY TURNPIKE	□	DAY	4	60		RICKER
	■	NIGHT	4	60		RICKER
SCHUYLKILL EXPRESSWAY, PHILA.	X	DAY	4	60	50(TK	COLEMAN
SHIRLEY HWY, ARLINGTON, VA.	o	DAY	6	50		COLEMAN
FAIRFAX CO.,VA.	⊕	DAY	4	50		BPR
LODGE EXPWY, DETROIT	▽	DAY	4	60	40	BPR
FORD EXPWY, DETROIT-MILLER RD.	*	D&N	6	55	40	MALO[A]
LONYO BR.	⊕	DAY	6	55	40	BPR
TELEGRAPH RD.	▣	DAY	6	55	40	BPR
HAMNAN RD.	▲	DAY	6	65		BPR
BELT RD.	●	DAY	4	65		BPR
	◉	DAY	4	65		BPR
DETROIT	◐	DAY	6	45		
EDENS EXPWY, CHICAGO-LINCOLN AVE.	◖	DAY	6	60		
GULF FWY, HOUSTON (ON-RAMPS CLOSED)	▫	DAY	6	50	40	KEESE
E-W FREEWAY, HOUSTON	▲	DAY	4	50		* KEESE
CALIF. FREEWAYS	◉	DAY	8	55		(FILM)
NIMITZ FWY, OAKLAND, CALIF.	△	DAY	6	65		KELL
	▲	NIGHT	6	65		KELL

[A] BASED ON 5 MIN EACH HOUR DURING ALL 24 HR
[I] OPTIMUM POINT
[2] 8 LANES

Figure 3.44. Specific reported speed-volume relationships per lane in one direction of travel under interrupted flow conditions on freeways and expressways.

of "average delay," rather than by obtaining space mean speed. Even this relationship is difficult to measure under field conditions, as arrival and discharge rates on a cycle-by-cycle basis vary widely (25). Figure 3.45 shows the relationships of average delay and a computed average speed to traffic volume at a pretimed traffic signal. This curve is based on a computer simulation of 160 hr of operation of an intersection of two-lane two-way streets with a 60-sec cycle and a 50 percent cycle split.

Despite the difficulties of field measurement, a Chicago study did find a typical speed-flow relationship for a composite of 37 test sections with parking permitted and again for a composite of seven test sections with parking prohibited, as shown in Figure 3.46 (22). Test sections averaged about one-half mile in length, with different signal spacings and with some range of values for other variables. By multiple correlation, it was found that average speeds were most responsive to the number of signals per mile. Because data were averaged for 1-hr
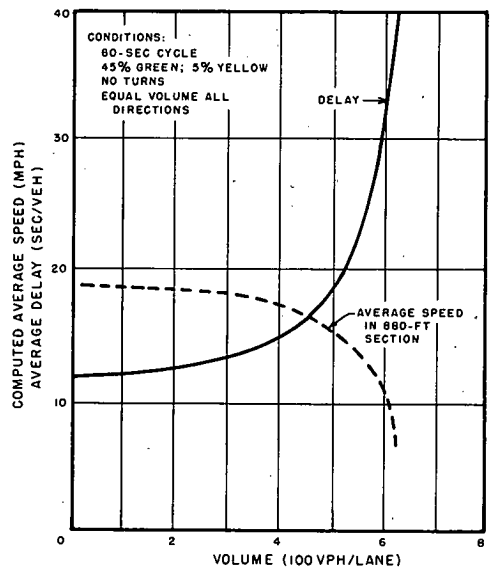
Figure 3.45. Computed average speed and average delay at simulated urban signalized intersection.
(Source: Ref. 47)

periods, the short-term speed and flow fluctuations were lost, and the resulting curves were best fitted to assumed linear curves. For the approximately 1,000 points represented by these curves, the correlation coefficients are 0.70 and 0.93, respectively.

Speed-flow curves found in a study in Charleston, W. Va., are shown in Figure 3.47. These curves are hand fit to data from 9 hr of observation of traffic on a 1,200-ft section upstream from an isolated urban traffic signal (26). The reverse curve indicates that critical density was regularly induced by the traffic signal. It should be noted that when the data were aggregated for different time periods, in increments of 6 min, the shape of the curves changed slightly. Here, then, is another caution in describing the speed-flow relationship: the time period of observation must be explicit. Although 1 hr is often a standard for discussion, more meaningful results sometimes are obtained from shorter time periods. The longer the period of observation, the less pronounced the effect of flow on space mean speed.

Even a study such as that just described must be interpreted with care, however. Its data were obtained from 7 AM to 11 AM on one day and from 2 PM to 7 PM on another. These increments include both peak-hour and off-peak traffic, hence contain a variety of types of drivers, some driving intently on rush-hour trips and others driving more casually on midday errands. As previously mentioned, the effect of this variable is not yet clear. Therefore, for some purposes it is desirable to separate the data into categories by time of day, to assure that differing human characteristics are not influencing the pure speed-flow relationship.

Finally, a London study (27) found curves for signalized sections which contain a definite "break" point. This is consistent with the hypothesis that at low flows maximum attainable speed depends almost solely on intersection signalization and midblock marginal frictions. Only at higher flows do stream frictions begin to restrict attainable speeds.
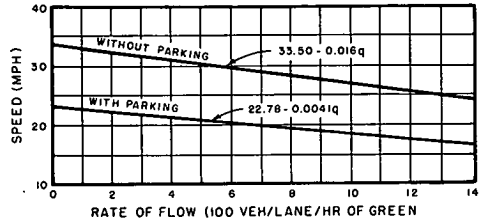
Figure 3.46. Speed-flow relationship for 37 test sections with parking and 7 test sections without parking.
(Source: Ref. 22)
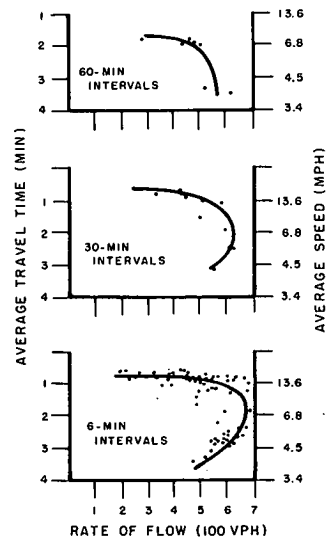
Figure 3.47. Travel time and average speed on signalized street in intermediate urban area.
(Source: Ref. 26)

Speed-Density Relationships

UNINTERRUPTED FLOW

Inasmuch as speed, density, and flow are interrelated, much of the previous discussion of the speed-flow relationship applies also to the speed-density relationship. Density is the number of vehicles in a particular length of roadway at a particular moment. Usually, it is expressed in vehicles per mile. Although an instantaneous value, it is also possible to average successive observations
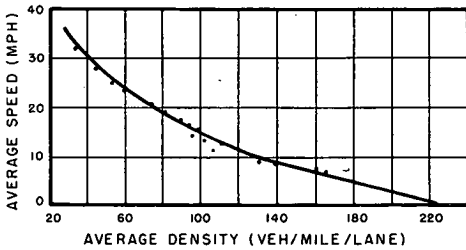
Figure 3.48. Speed-density relationship, Lincoln Tunnel, New York.
(Source: Ref. 29)
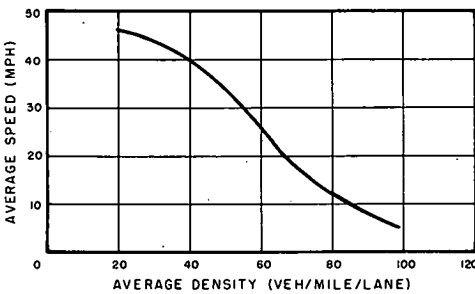
Figure 3.49. Speed-density relationship, Eisenhower (Congress St.) Expressway, Chicago.
(Source: Ref. 30)

over a period of time. Thus, if vehicle counts were made each minute for an hour, the arithmetic mean would express the average density for the hour. This differs from flow, which is a rate of movement per unit of time, the practical minimum time of observation being perhaps 1 min.

The speed-density relationship is similar to the speed-flow relationship in that in the upper range speed decreases with increasing flow and density. However, density continues to increase past the point of critical density, whereas flow decreases. This characteristic sometimes makes density a more advantageous speed predictor than flow.

Some of the earliest capacity studies, such as a 1934 study of rural roads in Ohio, found that there was a straight-line relationship between the average density of cars per mile and the space mean speed (28). A number of recent studies have more

intensively investigated the speed-density relationship under uninterrupted flows. In general, the speed-density relationships were described by mathematically fitting the best curves to observed data. Graphical plots used to describe some of these relationships are shown in Figures 3.48 through 3.52.

Figure 3.48 is a plot of average speed and density resulting from a least-squares fit to experimental data taken in the Lincoln Tunnel, New York (29). The points shown are the empirical data obtained by using 5-min averages to compute space mean speeds and mean densities. A high degree of correlation is noted.

Figure 3.49 shows a speed-density relationship for a section of the Eisenhower (Congress St.) Expressway in Chicago (30). Individual speeds and densities were measured within a 400-ft trap and averaged on a per minute basis.

A speed-density relationship for one location on the Hollywood Freeway in Los Angeles (31) is shown in Figure 3.50. Each plotted point represents 1 min of observations and the curve is included to indicate the average trend between speed and density.

Figure 3.51 represents data obtained in lane 1 upstream of a bottleneck produced by a narrow temporary bridge on the Merritt Parkway in Connecticut (32). Each point shown was determined from a 5-min sam-
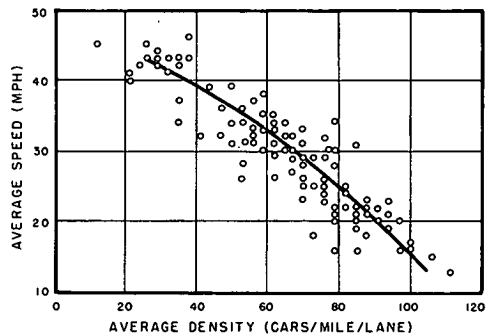
Figure 3.50. Speed-density relationship, Hollywood Freeway at Franklin West, Los Angeles, April 19, 1961.
(Source: Ref. 31)

ple of data, with density being computed by dividing the rate of flow by the mean speed. A straight line was mathematically fitted to the data points and a high coefficient of correlation was obtained.

Speed-density relationships for non-freeway facilities are shown by Figure 3.52 (33). The speeds used in these plots were not observed, but were computed from a linear regression equation based on observed volumes. It should be noted that if the speed-flow relationship is a straight line, the speed-density relationship will be nonlinear, and vice versa. Although experimental results have not determined which is the case, the weight of evidence appears to support a nonlinear speed-density relationship within a density range of 20 to 160 vehicles per mile. The difference is, perhaps, of greater theoretical than practical significance at present.

As a result of the previously cited work, it has been suggested that traffic flow can best be described in terms of three distinct zones—a zone of normal flow, a zone of
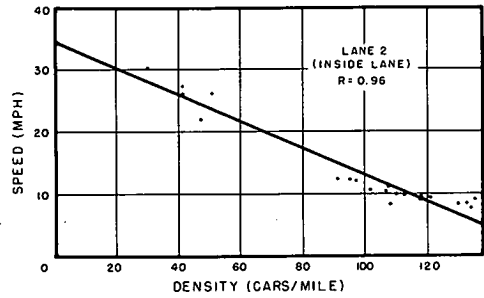
Figure 3.51. Speed-density relationship, Merritt Parkway, Conn.
(Source: Ref. 32)

unstable flow, and a zone of forced flow—each zone being specified in terms of probabilities, as in Figure 3.53 (35). When developed beyond the theoretical approach, a family of curves expressing speed-density and speed-flow relationships may well be a very practical and realistic method of summarizing traffic characteristics.
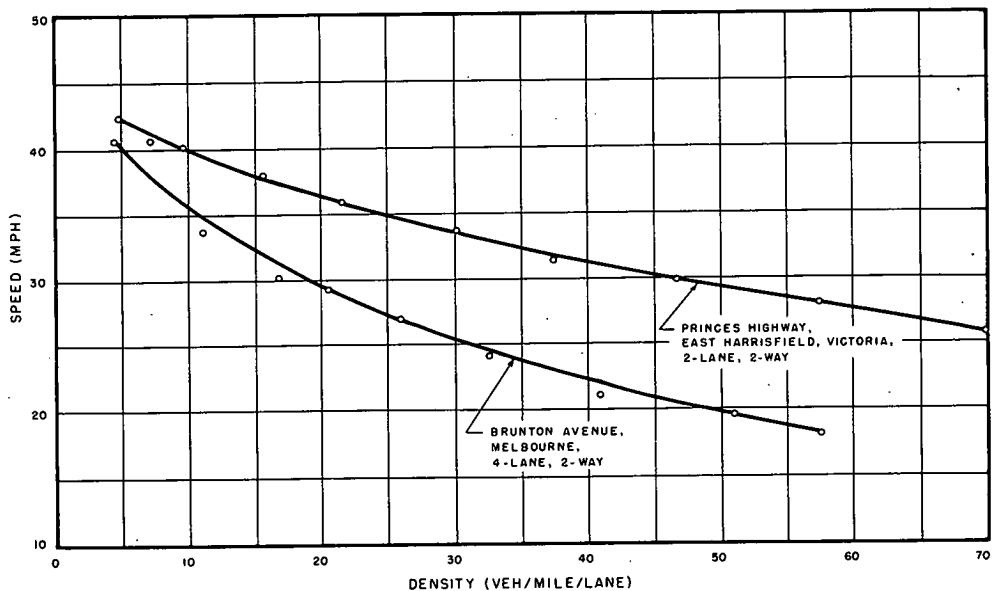
Figure 3.52. Speed-density relationship under uninterrupted flow conditions on two Australian highways.
(Source: Ref. 33, pp. 59 and 61)
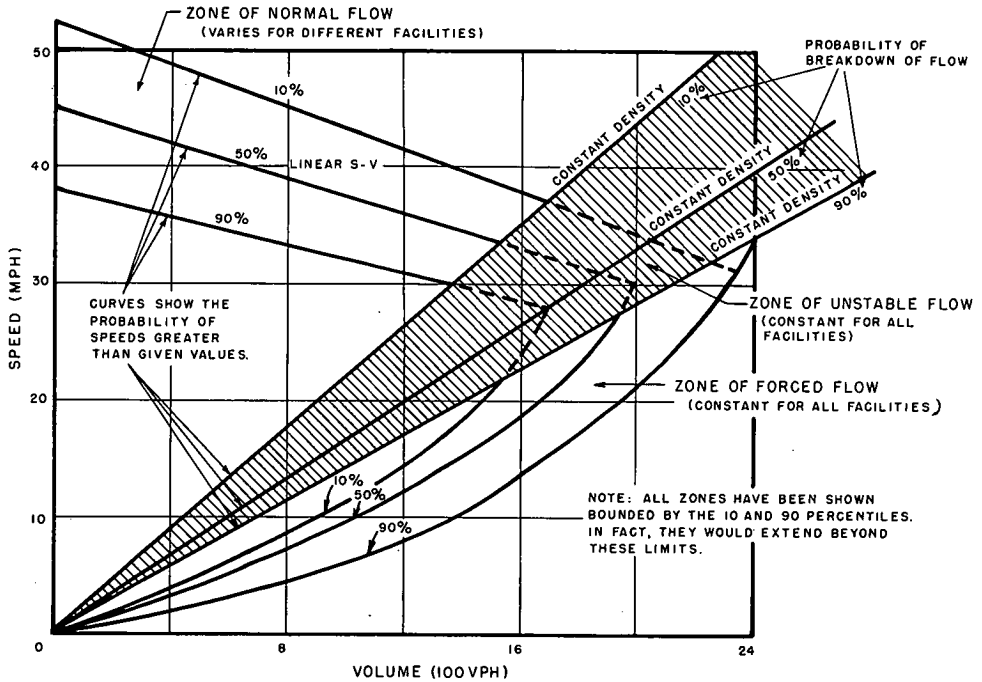
Figure 3.53. General speed-volume diagram (to demonstrate principles; not for use in specific problem applications).
(Source: Ref. 35)

INTERRUPTED FLOW

One of the difficulties in distinguishing between types of flow is that sooner or later all flows are interrupted. Thus, a freeway ends, a rural road enters a town, or a progressive signal system becomes unable to accommodate the demand. The careful investigator of an uninterrupted flow condition should indicate the length of the section studied and the distance in each direction to the first interruption of flow. Ordinarily, however, there is little difficulty in defining interrupted flow itself; it usually implies signalized or stop sign conditions.

The speed-density relationship under typical urban interrupted flow conditions is subject to many of the same problems of measurement previously discussed under the speed-flow relationship. However, a number of successful studies have been made which indicate that the relationship is very similar to that under uninterrupted flow con-

ditions. A Chicago study (36) found a straight-line relationship in a one-quarter mile arterial section in an intermediate-type area approaching a fixed-time signal, as shown by the curve marked Washington Blvd. in Figure 3.54.

Instantaneous densities at 1-, 2-, and 3-min intervals were correlated against test car space mean speeds obtained before, during, and after the density observations. Space mean speed and density (ahead of the test car) were highly associated ($r^2 = 0.89$); speed and flow were not ($r^2 = 0.04$). A New Haven study found a more steeply sloped speed-density relationship in a one-lane urban flow, with a correlation of $r^2 = 0.86$. The author, however, indicated a theoretical preference for a curvilinear boundary curve instead of the best-fit curve (37). Studies of one-way flow toward a bottleneck, which show characteristics of interrupted flow, have been made on the

Merritt Parkway and in New York's Lincoln Tunnel (37, 38). Both found speed-density boundary curves similar to those under un-interrupted flow conditions.

Flow-Density Relationships

The relationship between flow and density is curvilinear, as shown in Figure 3.55 (39).

Assuming constant speeds for a particular roadway, an increase in density results in a linear increase in flow, and vice versa. At some point as the density increases from zero, however, speed does decrease, as described in the preceding section, and the relationship becomes curvilinear. As the point of critical density is passed there is a decrease in flow despite the continued increase in density.

This is why complete congestion can occur when a highway is operating at or near its possible capacity. Let it be assumed that traffic on a road represented by Figure 3.39 has gradually increased to a volume of 2,000 vph per lane in one direction and the average speed has reduced to 30 mph, and that this traffic volume is approaching a temporary restriction where average speed must drop to 20 mph. The rate of flow of vehicles on the highway at the point where speeds are only 20 mph can only approximate 1,700 vph per lane. Therefore, vehicles would immediately start to accumulate at this point on the highway at the rate of
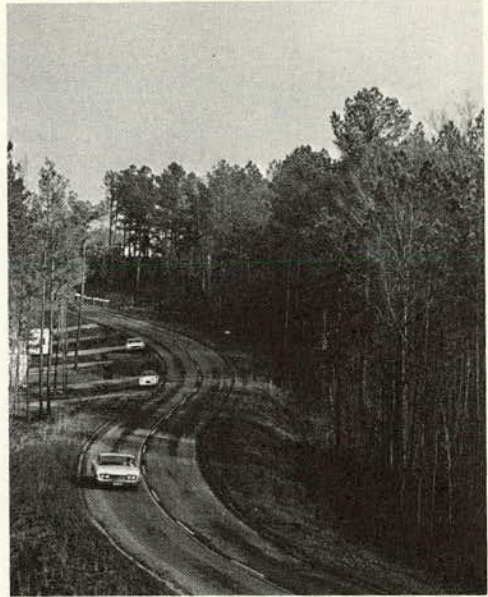
Typical rural two-lane highway carrying less than 1,000 vehicles per day.

300 per lane per hour, causing a sudden increase in density of traffic. If the approach flow of 2,000 vph per lane continued, even though the restricted condition existed for only a few seconds, some vehicles would be
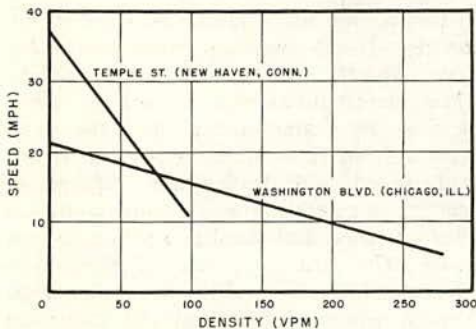
Figure 3.54. Speed-density relationship under urban conditions.
(Source: Refs. 36, 37)
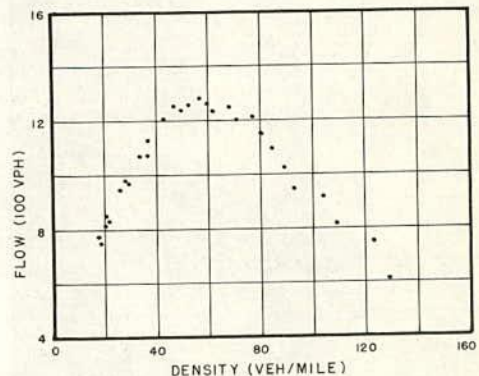
Figure 3.55. Example of flow-density relationship in limited-access traffic flow (Holland Tunnel, New York).
(Source: Ref. 39)

Directional distribution patterns on a freeway complex.

required to stop and the instantaneous traffic flow would immediately drop to zero at this point on the highway. The queue of vehicles at a standstill would continue to increase in length as long as the arrival rate of vehicles at the tail of the queue exceeded the departure rate at the head of the queue.

Even though the cause of the restriction lasted but a few seconds or minutes, additional vehicles might continue to become stopped for a considerable time after the cause of the restriction was removed. These vehicles would form a queue which would move down the highway in the direction opposite to that of traffic flow. Queues of vehicles at a standstill have been observed several miles from the scene of the original restriction, even though traffic was apparently operating in its normal manner between the queue and the place where the queue originally started to form. This phenomenon has been extensively studied, resulting in formulation of the theory of backward kinematic waves (40).

The curvilinear flow-density relationship appears common to both uninterrupted and interrupted flows. In signalized sections with moderately heavy flow there is an almost constant succession of wave movements due to artificial creation of critical densities at each red signal indication. Because of added time losses in acceleration and deceleration, it is likely that the flow-density curve for interrupted flow is well below that for uninterrupted flow.

Summary

This section has summarized some of the theoretical and observed relationships between speed, density, and rate of flow. Their interrelationships with capacity should be clear. By definition, capacity *is* the maximum rate of flow under stated conditions. Both speed and density are required to produce a rate of flow. Various combinations of speed and density produce various flows. The maximum rate of flow on a particular highway is at the point of critical density, which depends on the minimum headways that drivers find tolerable at particular speeds. Generalizing, it is found that the higher the type of highway, the shorter

these headways may be. Consequently, critical density occurs at somewhat different speeds and densities, depending on the type of highway.

Within certain limits the relationships explored in this section are found on most typical highways, bearing in mind the basic differences that distinguish uninterrupted and interrupted flows. By definition, such relationships are found wherever vehicles regularly follow one another in the same or adjacent lanes. Weaving sections, rotaries, and other types of maneuver areas are therefore exceptions. Also, within limits, these relationships hold true for any reasonably short periods of time, such as 1 min, 10 min, or 1 hr. They are sharpest and most meaningful for the shorter periods; data aggregated over periods of more than 1 hr tend to obscure short-run fluctuations.

The individual researcher and user of the manual should be aware of these inherent relationships in order to fully understand highway capacity. The references in this chapter represent a starting point. A vast amount of work has been accomplished in this field, and the serious student is well advised to study the literature in greater detail.

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CAPACITY AND LEVEL OF SERVICE

Chapter Three summarized available data on maximum observed volumes on different types of highways, and presented information on such traffic characteristics as variability of traffic flow and volume-speed-density relationships by type of highway. In this chapter, information in Chapter Three is applied in presenting the Committee's recommendations on the following:

1. Capacity, in numerical values, for various types of highways with uninterrupted flow under ideal conditions.

2. Levels of service, and criteria for identifying each of several levels, for various types of highways.

3. A generalized procedure for determining the level of service that will be obtained when a specific volume is carried over a section of highway under actual conditions.

Corrections for conditions which are not ideal, because of such factors as reduced widths, restricted sight distances, grades, and trucks, are described in Chapter Five. Applications of the generalized procedure for determining the level of service for different types of highways and streets are given in Chapters Nine and Ten.

CAPACITY FOR UNINTERRUPTED FLOW CONDITIONS

The maximum observed traffic volumes, as reported in Chapter Three, together with the results of speed-volume relationship studies also discussed in that chapter, have been used as a guide in establishing the numerical values of the capacity of different types of roadways for ideal conditions. The capacity of any individual section of roadway would vary from the maximum value of capacity for that type of roadway, depending on how its roadway and traffic characteristics vary from ideal conditions. For the purpose of

analysis, ideal conditions are defined as follows:

1. Uninterrupted flow, free from side interferences of vehicles and pedestrians.

2. Passenger cars only, in the traffic stream.

3. Traffic lanes 12 ft wide, with adequate shoulders and no lateral obstructions within 6 ft of the edge of pavement.

4. For rural highways, horizontal and vertical alinement satisfactory for average highway speeds of 70 mph or greater, with no restricted passing sight distances on two- and three-lane highways.

It is apparent that few roadway sections have all of these "ideal" conditions of operation. A few parkways built to high-type geometric design standards, with full control of access and carrying no commercial vehicles, may actually attain this status, and many modern level freeways come very close, meeting all criteria except the "all passenger car" requirement.

It is important to emphasize that "ideal" geometrics and traffic characteristics do not imply good operating conditions *per se.* Although ideal conditions do produce the highest volumes for any given level of service, operation at capacity, or maximum possible volume, will be unsatisfactory even under ideal conditions.

For multilane highways, the largest number of vehicles that can pass a point one behind the other in a single lane, under ideal conditions, averages between 1,900 and 2,200 passenger vehicles per hour. This represents an average maximum volume per lane sustained over the period of one hour, when all through lanes are considered in developing the average. Various studies have found higher lane volumes for specific lanes or for short time periods on multilane facilities, reaching the 2,400- to 2,500-vph

range, but they do not represent sustained volumes representative of all lanes. Where there are at least two lanes for the exclusive movement of traffic in one direction, and disregarding the distribution of traffic between lanes, *the capacity of a multilane highway under ideal conditions is considered to be 2,000 passenger vehicles per lane per hour.*

For two-lane, two-way highways, overtaking and passing maneuvers must be performed in the lane normally used by oncoming traffic. With traffic traveling in both directions, slower moving vehicles create gaps between vehicles that can be filled only by passing maneuvers, whereas these same gaps, if of sufficient length, provide passing opportunities for the opposing traffic. Travel during heavy-volume conditions on two-way, two-lane highways, therefore, oscillates between the formation of queues with gaps between, and the partial filling of these gaps by passing maneuvers.

Studies have shown that with traffic evenly divided by direction vehicular operation is sufficiently restricted to limit the flow in each direction to 1,000 passenger vehicles per hour. At the other extreme, when almost all traffic is moving in one direction, the one lane can be kept completely filled by passing maneuvers. The capacity under these conditions is limited to the number of vehicles that can crowd into one traffic lane, because the other traffic lane must still be reserved for opposing traffic. *The capacity of a two-lane, two-way roadway under ideal conditions is, therefore, 2,000 passenger vehicles per hour, total, regardless of distribution by direction.*

Traffic operation on typical three-lane, two-way highways is similar to two-lane highways, except that an additional lane is provided for passing maneuvers in either direction. With traffic evenly divided by direction, the capacity under ideal conditions would approach the number that can crowd into two traffic lanes, because the center lane can be utilized for passing maneuvers to fill the long gaps between vehicles. Conversely, predominant movement in one direction will preempt the center lane so the characteristics of flow become similar to those in one direction on a four-lane highway. *The capacity under*

TABLE 4.1—UNINTERRUPTED-FLOW CAPACITIES UNDER IDEAL CONDITIONS

HIGHWAY TYPE	CAPACITY (PASS. VPH)
Multilane	2,000 per lane
Two-lane, two-way	2,000 total both dir.
Three-lane, two-way	4,000 total both dir.

ideal conditions for a three-lane, two-way roadway approaches 4,000 passenger vehicles per hour, regardless of distribution by direction. This figure is reduced substantially by poor roadway alinement and profile. A single restrictive sight distance will restrict the capacity of a three-lane, two-way roadway to 2,000 vph in one direction, with directional distribution determining the total capacity.

Currently, there is increasing use of lane control devices on the center lane of remaining three-lane highways, either to make it a reversible lane or to reserve it for left turns only, in both directions. Insufficient data have been gathered thus far to permit specific capacity values for these situations.

Capacities for all basic highway types for uninterrupted flow under ideal conditions are summarized in Table 4.1.

It must be remembered, however, that these values were determined from studies of many highways under a variety of conditions. In all cases it would be impossible to state that the volume measured was the absolute maximum that could be carried, inasmuch as maximum volumes observed at different times at one point will show a range of values. Rather, each capacity value given in Table 4.1 should be considered as the average maximum volume, or a maximum volume that has a reasonable expectation of occurring frequently on the particular type of highway under ideal conditions.

In this connection it should be recognized that occurrences such as minor accidents or vehicle breakdowns, which are often referred to during traffic studies as "abnormal" or "unusual," actually may be

The capacity of a two-lane, two-way roadway under ideal conditions is 2,000 passenger vehicles per hour, total.

The capacity of a multilane highway under ideal conditions is 2,000 passenger vehicles per lane per hour.

quite common on heavily-used highways, possibly occurring several times in any given section during a typical peak period. It would be unwise to consider the rare case when no such incident occurred as the controlling case; a more realistic maximum is required.

In addition to the existence of ideal conditions, three other factors are implied. These should be restated as essential for the foregoing values to be attained in practice. First, there must exist immediately upstream from the roadway section provision for and a traffic demand equal to or greater than this capacity. Second, the roadway downstream from the section being studied must be of sufficient capacity to carry the traffic away. Finally, outside influences such as weather must allow the maximum capabilities to be utilized.

The values in Table 4.1, although not directly applicable to many highways, provide the basis for the succeeding capacity and level-of-service analysis techniques. The need for adjusting these capacities to actual roadway conditions is discussed in Chapter Five.

CAPACITY FOR INTERRUPTED FLOW CONDITIONS

Unlike uninterrupted flow, few broad criteria can be described for interrupted flow. It is not feasible to define fundamental capacities under ideal conditions, because too many variables are involved. Rather, any examination of interrupted flow requires detailed study of the elements producing the interruptions. Although any signalized intersection is obviously such an element, vari-

ous midblock interruptions may be equally significant.

Generally speaking, the following two basic limitations can be established: (1) Rarely does a traffic lane on an urban arterial carry volumes at a *rate* greater than 2,000 passenger cars *per hour of green signal indication,* even with ideal signal progression; and (2) a line of vehicles, all of which are stopped by an interruption, will only rarely move away from the interruption at a *rate* greater than 1,500 passenger cars per lane per hour, during those periods when the interruption is not in effect.

It is essential to note that these values are *rates,* not volumes. Thus the values given are measures of the maximum that would pass if sufficient periods of moving traffic were summed in the interrupted flow environment to total 60 min. They do not represent actual volumes per clock hour, which typically are considerably less. Thus, these values cannot be used as "rules of thumb" in the same sense as can the uninterrupted flow capacities.

LEVELS OF SERVICE

When the traffic volume equals the capacity of a highway, operating conditions are poor, even under ideal roadway and traffic conditions. Speeds are low, with frequent stops and high delay. In order that a highway provide an acceptable level of service to the road user, it is necessary that the service volume be lower than the capacity of the roadway. The maximum volume that can be carried at any selected level of service is referred to as the "service volume" for that level. Several such levels are specifically defined in this manual.

The individual road user has little realization of the volume level itself, but he is aware of the effect of high volume on his ability to travel on a street or highway with reasonable speed, comfort, convenience, economy and safety. Thus, factors which might be considered in evaluating level of service include the following:

1. Speed and travel time. This not only includes the operating speed, but also the overall travel time utilized in traversing a section of roadway.

2. Traffic interruptions or restrictions. This includes the number of stops per mile, the delays involved, and the magnitude, frequency, and suddenness of speed changes necessary to maintain pace in the traffic stream.

3. Freedom to maneuver. This considers the amount of freedom to maneuver to maintain desired operating speeds.

4. Safety. This includes not only accident rates, but also potential hazards.

5. Driving comfort and convenience. This considers roadway and traffic conditions as they affect driving comfort, and also considers the degree to which the service provided by the roadway meets the convenience standards of the driver.

6. Economy. This considers the cost of operating the vehicle on the highway.

Desirably, all of these factors should be incorporated in a level-of-service evaluation. As yet, however, there are insufficient data to determine either the values or the relative weights of certain of the six factors listed.

After careful consideration, the Committee has selected travel speed as the major factor to use in identifying the level of service. The Committee also uses a second factor—either the ratio of demand volume to capacity or the ratio of service volume to capacity, depending on the particular problem situation—in making this identification. Although the recommended level-of-service scales may not include all of the factors considered desirable, the use of these two factors is considered to represent a practical approach based on past and present experience.

In practice, the second factor is referred to as the "v/c ratio." In problems where demand and capacity are known and the level of service is desired, v primarily represents the demand. It also, of course, represents a service volume, but only coincidentally would it represent a controlling value marking the limiting service volume of a defined level of service. Usually, it would fall somewhere within a defined level-of-service range. On the other hand, in a case where capacity and a required level of service are specified v represents the computed

limiting service volume that can be handled at that level of service. Throughout this text, then, v may represent either demand volume or service volume, depending on the circumstances in which it is used.

Because design speed can vary considerably by type of highway, it is quite possible to obtain low operating or overall travel speeds on some highways because of physical design features, rather than traffic characteristics, regardless of the volume of traffic carried. Thus, a single level-of-service scale applying to all types of streets and highways is not considered feasible. Rather, separate recommended level-of-service scales are established in this manual for several different major highway types, and related scales are presented for certain highway elements. Even within a given highway type, variations in design standards will prevent some from offering the better levels of service.

Travel speed, used as one measure of level of service, may be either an operating speed or an average overall travel speed, depending on the type of highway. Operating speeds are used for those types of highways carrying generally uninterrupted flow; these are typically found in rural areas. Average overall travel speed is utilized for urban arterial and downtown streets, and interrupted flow generally, because this is the type of speed data normally obtained in urban areas.

The operating speed provides an indication of overall performance on a roadway. The additional evaluation of the volume-to-capacity ratios provides some indication of traffic densities and freedom to maneuver.

Each level of service should be considered as a *range* of operating conditions bounded by values of travel speed, and by volume-to-capacity ratios. Wherever speed and service volume values are given, to identify the limits of a level, they are considered to be, respectively, the limits representing the lowest acceptable speed and highest acceptable volume of a level-of-service range. It is essential to remember, however, that the identified level's range extends from the limits of the next higher level. When speeds are higher and service volumes are lower than the values given, operations are equal to or better than that level of service. As

traffic density increases and quality of service falls, only coincidentally will both limits be reached simultaneously; usually, one or the other of the limits will govern in any particular case. Once either limit is passed, service will drop to the next level.

Based on the previous discussion, the following criteria thus have been established for determining capacity and level-of-service relationships:

1. Volume and capacity are expressed in numbers of passenger cars per hour for subsections of each section of roadway. Demand volume and capacity may vary considerably along a section of roadway, and average values for an entire long section often may not adequately represent the actual conditions at all points within that section. The degree of detail necessary in dividing any particular section into subsections for separate examination will, of course, depend on the nature of the study.

2. Level of service, strictly defined, applies to a section of roadway of significant length. Such a section may have variations in operating conditions at different points or over subsections throughout its length, due to changes in demand volume or capacity. Built-in variations in capacity result from varying conditions along the roadway, such as changes in width, or presence of grades, ramp terminals, weaving areas, restricted lateral clearances, and intersections. Variations in volume result from varying amounts of traffic entering and leaving at points irregularly spaced along the roadway. The section level of service must, within limits, take into account the overall effect of these point and subsection limitations on the entire section. Therefore, for computational purposes, certain point or subsection equivalents to the more broadly defined levels of service must also be considered.

3. Analysis of volume and operating speed, or average overall travel speed, is made for each point or subsection of the highway having relatively uniform conditions. The weighted operating speed, or average overall travel speed, is then determined for the entire section, and a corresponding level of service is identified.

4. Elements used to measure capacity and levels of service are variables whose values

or categories are easily determined from available data. For capacity these include roadway type, geometrics, average highway speed, traffic composition, and time variations in volume. For level of service, additional elements used include speed and volume-to-capacity ratios.

5. For practical use, values of speed and volume-capacity ratio which define levels of service are established for each of the following types of facilities:

(a) Freeways and other expressways.
(b) Other multilane highways.
(c) Two- and three-lane highways.
(d) Urban arterial streets.
(e) Downtown streets (approximate only).

Related levels of service are established for several point elements, including intersections, ramp junctions, and weaving sections.

6. Criteria selected for practical application in identifying levels of service for various types of highways are given in Table 4.2.

OPERATING CONDITIONS FOR LEVELS OF SERVICE

Six levels of service have been selected by the Committee for application in identifying the conditions existing under various speed and volume conditions on any highway or street. It should be noted that other intermediate levels may be established by other jurisdictions for specific conditions. The descriptions here relate to uninterrupted flow and are broadly generalized; definitive values are given in Chapters Nine and Ten for each type of highway. These levels of service, designated A through F, from best to worst, cover the entire range of traffic operations that may occur. On many specific streets and highways, the better levels cannot be attained.

Traffic operational freedom on a highway of a particular type is considered equal to or greater than level of service A, B, C, or D, as the case may be, when specified values of the two separate conditions previously described are met. These conditions require that: (1) operating speeds or average overall speeds be equal to or greater than a standard value for the level considered, and (2) the ratio of the demand volume to the capacity of any subsection not exceed a standard value for that level. Level of service E describes conditions approaching and at capacity (that is, critical density). Level F describes conditions under high-density conditions when speeds are low and variable; it is not effectively described by combinations of speed and volume-to-capacity ratios, because these may vary widely.

Level of service A describes a condition of free flow, with low volumes and high

TABLE 4.2—ELEMENTS USED TO EVALUATE LEVEL OF SERVICE

ELEMENT	FREEWAYS	MULTI- LANE HIGHWAYS	TWO- AND THREE- LANE HIGHWAYS	URBAN ARTERIALS	DOWN- TOWN STREETS
Basic elements					
Operating speed for section	X	X	X		
Average overall travel speed				X	X
Volume-to-capacity ratio:					
(a) Most critical point	X	X	X	X	
(b) Each subsection	X	X	X	X	
(c) Entire section	X	X	X	X	
Related elements					
(a) Average highway speed	X	X	X		
(b) Number of lanes	X				
(c) Sight distance			X		

speeds. Traffic density is low, with speeds controlled by driver desires, speed limits, and physical roadway conditions. There is little or no restriction in maneuverability due to the presence of other vehicles, and drivers can maintain their desired speeds with little or no delay.

Level of service B is in the zone of stable flow, with operating speeds beginning to be restricted somewhat by traffic conditions. Drivers still have reasonable freedom to select their speed and lane of operation. Reductions in speed are not unreasonable, with a low probability of traffic flow being restricted. The lower limit (lowest speed, highest volume) of this level of service has been associated with service volumes used in the design of rural highways.

Level of service C is still in the zone of stable flow, but speeds and maneuverability are more closely controlled by the higher volumes. Most of the drivers are restricted in their freedom to select their own speed, change lanes, or pass. A relatively satisfactory operating speed is still obtained, with service volumes perhaps suitable for urban design practice.

Level of service D approaches unstable flow, with tolerable operating speeds being maintained though considerably affected by changes in operating conditions. Fluctuations in volume and temporary restrictions to flow may cause substantial drops in operating speeds. Drivers have little freedom to maneuver, and comfort and convenience are low, but conditions can be tolerated for short periods of time.

Level of service E cannot be described by speed alone, but represents operations at even lower operating speeds than in level D, with volumes at or near the capacity of the highway. At capacity, speeds are typically, but not always, in the neighborhood of 30 mph. Flow is unstable, and there may be stoppages of momentary duration.

Level of service F describes forced flow operation at low speeds, where volumes are below capacity. These conditions usually result from queues of vehicles backing up from a restriction downstream. The section under study will be serving as a storage area during parts or all of the peak hour. Speeds are reduced substantially and stoppages may
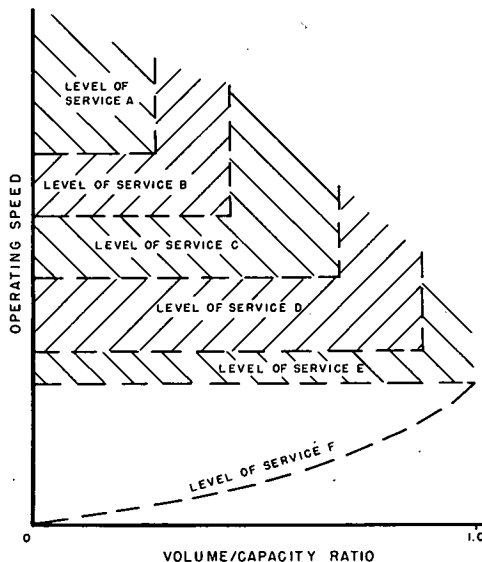
Figure 4.1. General concept of relationship of levels of service to operating speed and volume/capacity ratio. (Not to scale.)

occur for short or long periods of time because of the downstream congestion. In the extreme, both speed and volume can drop to zero.

These levels of service are depicted conceptually in Figure 4.1.

SUMMARY OF PROCEDURES

This chapter presents only the broad generalized concept for determination of the attainable level of service for a typical section of highway having uninterrupted flow. Detailed procedures for determination of capacity, service volumes, and level of service are presented in Chapters Nine and Ten. The basic concept involves the following steps:

1. Subdivide the roadway section into subsections having reasonably uniform conditions from the standpoint of capacity. Also, identify for possible separate detailed analysis points which may be critical bottleneck locations.

2. Determine, for each subsection and critical point, the capacity, demand volume,

Level of service concept on a multilane freeway as viewed from above. Note center roadway.
Center left: *level B, stable flow, few restrictions on operating speed;* Lower left: *level C,*
stable flow, higher volume, more restrictions on speed and lane changing; Upper right:
level D, approaching unstable flow, little freedom to maneuver, condition tolerable
for short periods; Center right: *level E, unstable flow, lower operating speeds than*
level D, some momentary stoppages; Lower right: *level F, forced flow*
operation at low speeds, highway acts as storage area, many stoppages.

A

E

F

and volume-to-capacity ratio. Capacity is the value obtained by applying appropriate reduction factors, described in Chapter Five, to the ideal capacity for that highway type, to correct for physical factors (such as widths, clearances, grades) and traffic factors (such as commercial vehicles) less than ideal.

For refined determination of level of service, a "base volume" must be used instead of actual capacity, to reflect the effect of correction factors that are different for levels of service than for capacity. It is computed in the same way as corrected capacity, except for use of the different correction values.

3. For each subsection, use the resulting volume-to-capacity ratio, v/c, or, more precisely, volume-to-base volume ratio to determine an operating speed for that subsection. This operating speed is obtained from the appropriate table or curve selected from the typical speed-volume relationships given in Chapters Nine and Ten, taking into account the type of highway and its average highway speed. If desired, the level of service for each particular subsection can be established from these measures, using criteria in Chapters Nine and Ten.

(Note: Operating speed is not a measure commonly recorded in traffic studies, because it can be obtained only by actual timed runs of vehicles driven as fast as possible, without exceeding the design speed, at various volume levels. Where, however, it is known from such observations, it should be used directly.)

4. Determine the overall level of service of the several subsections combined. First, compute the weighted averages of operating speeds and of v/c ratios for the entire section, as described in Chapters Nine and Ten. Then use the overall weighted average operating speed and v/c ratio thus computed to determine the resulting overall level of service for the section, based on standards presented in Chapters Nine and Ten for the appropriate type of highway.

5. Check the most critical v/c ratio or ratios in the section to make sure that capacity is not exceeded at any point. Where controlling level-of-service or v/c ratio limits have been established for all the points along a section, check further to make sure that these are not exceeded at any point.

A simple example of the general use of the procedure follows, as applied to a rural roadway section with given physical and traffic conditions and an estimated demand volume.

EXAMPLE OF LEVEL-OF-SERVICE DETERMINATION FOR UNDIVIDED FOUR-LANE HIGHWAY WITHOUT ACCESS CONTROL UNDER UNINTERRUPTED FLOW CONDITIONS

ELEMENT	IDEAL GEOMETRICS SECTION	CURVE SECTION	RESTRICTED WIDTH SECTION	GRADE SECTION.
Length (mi)	2.0	0.5	1.5	1.0
Avg. highway speed (mph)	70	50	60	70
Capacity, ideal [a]	4,000	4,000	4,000	4,000
Reduction factor [b]	0.95	0.90	0.85	0.70
Capacity, actual [a]	3,800	3,600	3,400	2,800
Demand (vol./hr) [a]	2,000	2,000	2,000	2,000
v/c Ratio	0.53	0.56	0.59	0.71
Weighted avg. v/c ratio	←		0.59	→
Operating speed [c] (mph)	51	38	42	45
Wtd. avg. operating speed (mph)	←		45.8	→

[a] One direction.

[b] See Chapter Five.

[c] Operating speed obtained for each section of roadway from curves showing relation between operating speed and v/c ratio for the pertinent type of highway with that average highway speed.

Results

This analysis determines an overall operating speed of 45.8 mph, an overall v/c ratio of 0.59, and critical v/c ratio of 0.71, for use in establishing the level of service associated with the 2,000-vph demand volume.

Criteria presented in Chapter Ten show that level of service C is thus indicated, and that the grade does not produce a drop into a poorer level of service.

The service volume and level-of-service concepts for uninterrupted flow may at first seem quite different from earlier methods, but fundamentally there is little change. The measures used in both cases are a volume less than capacity, and a related operating speed. The earlier procedures offered a choice of two levels below capacity—namely, "practical-urban" and "practical-rural"— each of which, by definition, was associated with an operating speed range pre-established by the Committee as appropriate nationally. Separate volume scales were established for two-lane, three-lane, and multilane roads, but speed levels were approximately equal.

The new procedures offer a choice of four levels below capacity, each of which is related to an operating speed; these levels offer more freedom to the local administrator or engineer to select that type of operation most suitable for his local conditions. Separate scales now are established in terms of both volume and speeds, for levels of service on freeways, multilane highways without access control, and two-lane roads. It should be noted that for freeways and other multilane highways, levels of service can be determined separately for each direction of flow, whereas on two-lane highways only the overall level for the total two-way flow can be established because only two-way capacity criteria are available.

Levels of service, then, incorporate no radically new concept. Rather, they refine the previous procedures to incorporate a new degree of flexibility for local applications. The need for such flexibility has become increasingly apparent over the years, particularly in connection with analyses of benefits provided in relation to the costs of providing them. Highway planning, financing, construction, and operation have become too complex and interrelated subjects to permit the Committee to attempt to define all-inclusive "practical" levels equally suitable throughout the nation, or even the world, as it did in 1950.

Utilizing the refined procedures in this manual, designers and traffic engineers in specific localities are encouraged to develop their own charts and tables for capacity and service volume determinations, based specifically on local traffic, environmental and geometric characteristics. Such charts may permit the bypassing of several of the steps contained herein.

FACTORS AFFECTING CAPACITY AND SERVICE VOLUMES

Chapter Four provides fundamental capacity values for various types of highways and describes levels of service under ideal conditions. It is seldom, however, that all roadway and traffic conditions which affect capacity are ideal. Therefore, determination of service volumes for most highway sections requires application of adjustment factors described in this chapter.

The determination of a service volume, of course, first depends on choice of the level of service desired for the highway under consideration, as discussed in detail in Chapters Nine and Ten. Adjustment factors for certain effects apply equally to capacity and the several levels of service, whereas those for other effects differ depending on the level to which they are to be applied.

Factors affecting capacity and level of service are described under two categories —roadway factors and traffic factors. In some cases, the two categories are interrelated. For example, most grades would not affect capacity appreciably were it not for trucks in the traffic stream. Conversely, the effect of trucks on capacity is much greater on long, steep upgrades than on level sections.

Not all of the factors affecting capacity and level of service have been fully evaluated as yet. This is particularly true of the level-of-service factors, because the overall concept is relatively new. Further research is essential in some areas before firm numerical adjustment values can be assigned, or before refined separate adjustments for the several different levels of service can be developed. The figures included here represent the best estimate of these factors that can be presented as guides at this time.

It is significant to note that although these factors are intended to reflect the influence of certain variables on the capacities and service volumes of highways, they also indirectly reflect the degree of safety. In nearly every case, an element which reduces the amount of traffic that can be carried also creates greater accident potential. However, other elements which do not affect capacity may nevertheless affect safety.

ROADWAY FACTORS

Restrictive physical features incorporated into the design of a section of roadway have an adverse effect on its capacity and service volumes. Such elements are called "roadway factors" in this manual. Roadway factors discussed in this chapter include: lane width, lateral clearance, shoulders, auxiliary lanes, surface conditions, alinement, and grades.

Lane Width

Narrower lanes have a lower capacity under uninterrupted flow conditions than the 12-ft lanes which the Committee has accepted as the defined ideal. On a two-lane highway, a vehicle performing a passing maneuver occupies the lane normally used by traffic traveling in the opposite direction for a longer period when the lanes are narrow than when they are wide. On multilane highways, more vehicles encroach on adjacent lanes when the lanes are narrow than when they are wide, in effect occupying two lanes rather than one at such times.

Table 5.1 gives the capacities of lanes from 9 to 12 ft in width expressed as a percentage of the capacity of a 12-ft lane. These percentage factors are applicable only under uninterrupted flow conditions. The effect of lane width on capacities of inter-

sections where stop-and-go operation prevails is discussed in Chapter Six.

Table 5.1 is presented for information only. Its use is not required for the determination of roadway capacity or service volumes, as described in the procedures chapters of this manual, inasmuch as the combined effects of lane width and restricted lateral clearance are there presented as single adjustment factors.

Although the table shows only the effect of narrower lanes on capacity, such restricted lanes also adversely affect driver comfort and increase potential hazard.

Lateral Clearance

It is believed that mountable curbs and vertical curbs 6 in. or less in height have insignificant influence on traffic operations. However, other lateral obstructions (such as retaining walls, abutments, signposts, light poles, and parked cars) located closer than 6 ft from the edge of a traffic lane reduce its effective width. Table 5.2 shows, as an example, how restricted lateral clearances on both sides of a 24-ft, two-way pavement carrying uninterrupted flow reduce its effective width. For instance, a section of 24-ft pavement with a bridge truss at the edge has the same effective width as a 17-ft pavement with no obstructions on either side closer than 6 ft.

Judgment must be exercised when evaluating the effects of lateral restrictions on the level of service provided by a given section of highway where the restrictions are not continuous throughout its length. Even one lateral restriction may cause a bottleneck and thereby directly affect the capacity of the entire section, but operation at lower volumes (better levels of service) may not be seriously affected.

Continuous obstructions (such as median barriers, guardrails on long viaducts, and high barrier curbs) may have less adverse effect on effective pavement width than intermittent, short obstructions, because drivers become accustomed to them. For example, one study made on a freeway of the results of erecting a barrier fence in a 4-ft raised median with 6-in. curbs showed that the barrier had no significant effect on vehicle placement (*1*).

TABLE 5.1—EFFECT OF LANE WIDTH ON CAPACITY FOR UNINTERRUPTED FLOW CONDITIONS

LANE WIDTH (FT)	CAPACITY (% OF 12-FT LANE CAP.)	
	2-LANE HIGHWAYS	MULTILANE HIGHWAYS
12	100	100
11	88	97
10	81	91
9	76	81

TABLE 5.2—EFFECTIVE ROADWAY WIDTH DUE TO RESTRICTED LATERAL CLEARANCES UNDER UNINTERRUPTED FLOW CONDITIONS

CLEARANCE FROM PAVEMENT EDGE TO OBSTRUCTION, BOTH SIDES (FT)	EFFECTIVE WIDTH OF TWO 12-FT LANES (FT)	CAPACITY OF TWO 12-FT LANES (% OF IDEAL)
6	24	100
4	22	92
2	20	83
0	17	72

As a "rule of thumb," high barrier curbs can be identified as those high enough to damage the body and fenders of vehicles coming in contact with them. (The influence of curbs higher than 6 in. but lower than this level remains questionable.) Data (*2*) on the lateral placement of vehicles with respect to high barrier curbs show that drivers shy away from them, but that the average clearance allowed is probably somewhat greater where the curb is first introduced than along sections where it has been continuous for some distance. The extent of shying away is not known precisely, but it is known that the lateral placement of vehicles varies with the curb height and steepness and the position of other obstructions outside the curb. Present indications

Lane widths of less than 12 ft, plus restrictive lateral clearances, substantially reduce capacity.

are that an introduced barrier curb has the full influence indicated by the lateral clearance factors where first encountered, but that if it is continuous its effect gradually becomes less as drivers adjust to its presence.

It should be remembered that lateral clearances which are "ideal" from a capacity standpoint—that is, 6 ft or greater—are not necessarily adequate from a safety standpoint. In modern high-type designs, therefore, safety considerations generally govern the clearance finally established.

Table 5.2, like Table 5.1, is presented for information only; it is not used in actual problem solutions, because the combined adjustments discussed in the following are used instead.

Lane Width and Lateral Clearance (Combined)

In practice, there is seldom a need to know the individual effects of lane width and of lateral clearance, because they are largely interrelated. Therefore, for convenience in actual problem solutions their combined effects are consolidated into single adjustment factors, which are given in Tables 9.2, 10.2, and 10.8. These tables cover a range

of roadway widths from 9 to 12 ft, and a range of lateral clearances from 0 to 6 ft. They present, where appropriate, separate adjustments for cases where obstructions exist on only one side of the roadway and for cases where they exist on both sides at equal distances.

In cases where obstructions exist on both sides, but at differing distances, interpolation between factors is acceptable. For instance, given a setback of 4 ft on one side and 2 ft on the other on a four-lane highway with 10-ft lanes, an average of the tabulated factors for a setback of 4 ft on both sides and 2 ft on both sides would be obtained.

Shoulders

Adequate shoulders are essential if the capacity provided by the traffic lanes is to be maintained continuously. Without a place of refuge outside the traffic lanes, one disabled vehicle can reduce the capacity of a highway by more than the capacity of one lane, particularly if the lanes are less than 12 ft wide. The disabled vehicle blocks the lane occupied, and, in addition, reduces the capacity of adjoining lanes whenever vehicles must merge into fewer lanes at speeds below that at which capacity occurs for the highway in question.

For example, the capacity of a traffic lane on a certain multilane highway may be attained at 30 mph. However, if it is adjacent to a blocked lane and speeds in it fall to 20 mph, possibly only 85 percent of its capacity may be realized, and at 10 mph the lane may be able to carry only about 50 percent of its 30-mph capacity. Therefore, a minor accident which causes a reduction in speed can cause complete congestion on a highway operating at or near its capacity. This loss of traffic-carrying capability due to speed reduction also explains why accidents which do not block the traffic lanes often cause congestion due to excessive slowing by curious drivers.

In addition to maintaining the capacity of a highway by providing refuge for disabled vehicles, adequate shoulders also in some cases increase the effective width of traffic lanes. For lanes less than 12 ft wide, shoul-

Adequate shoulders are essential to maintain capacity of traffic lanes. Continuous shoulders are provided on both sides of this 4-lane non-controlled-access highway.

ders which are paved or which are treated with bituminous materials for a width of 4 ft or more increase the effective width of the adjacent traffic lanes by 1 ft.

Auxiliary Lanes

An auxiliary lane is the portion of the roadway adjoining the through traveled way used for parking, speed change, weaving, turning, storage of turning vehicles, separation of slow-moving vehicles on long, steep upgrades, and other purposes supplemental to through traffic movement. Auxiliary lanes generally are provided to permit effective utilization of the capacity of, and to improve the quality of service on, the through traffic lanes. As such, they often serve to prevent development of bottlenecks.

PARKING LANES

Obviously, a pavement of any width suffers a loss in capacity if parking is present. This is true even if only a few vehicles park at any one time, legally or illegally. Therefore, for capacity analysis purposes, primary consideration must be given to the actual parking conditions along the street, rather than to parking regulations.

Where parked vehicles must be accommodated along the highway, the provision of parking lanes greatly reduces the adverse effect of the parking on the highway's capacity. Parking lanes may also serve as shoulders for the parking of disabled vehicles if space is available where a particular breakdown occurs.

However, it is not correct to state that any parking lane wide enough to accommodate the parked vehicles physically will prevent all capacity loss due to parking. This is because parking, like any other obstruction, has an influence range extending beyond its physical limits. For example, assuming an 8-ft parking lane, and applying uninterrupted flow principles, the parked cars would constitute an obstruction located about 1 ft from the edge of the traffic lane, with restrictive effects as just discussed, even if it were assumed that they were parked very close to the curb or lane edge.

It is not common practice to provide for parking lanes on rural highways. If a spe-

cific problem occurs where a parking lane is added to a rural highway, the effect on capacity can best be handled by considering the parked vehicles to constitute a lateral obstruction. On signalized urban streets, where lateral clearance corrections are not used directly, it appears that a row of parked vehicles takes up an effective width of some 12 to 14 ft, in the average case, though the effect may vary quite widely in specific cases. No separate adjustment need be made for this effect, however, because parking conditions are incorporated as a basic element in the intersection capacity determination procedures described in Chapter Six. Correct use of these procedures automatically makes proper allowance for parking.

SPEED CHANGE LANES

Deceleration lanes permit traffic leaving a highway to decelerate, after moving off the through pavement, from its normal operating speeds in the through lanes either to comfortable and safe exit roadway speeds or, where necessary, to a full stop, before moving onto the adjacent connecting highway. Thus, during normal operation they eliminate the need for excessive slowing on the through pavement, which is one cause of congestion at exits from high-volume highways.

Acceleration lanes permit entering traffic to accelerate to speeds close to normal operating speeds on the through highway before entering the through traffic lanes and to adjust their speeds as necessary to match openings or gaps, so as to merge smoothly into the through traffic stream. A smooth or free-flowing merging condition contributes materially to the quality of service provided, both for through and entering traffic, at an entrance to a highway.

The influences of traffic merging into and diverging from the main traffic stream are too complex to be represented by simple correction factors. Complete capacity analyses are needed at these locations. The reader is referred to Chapter Eight for detailed discussion of capacity analyses at ramp entrances and exits.

TURNING AND STORAGE LANES

Separate turning lanes apart from the through pavement may substantially improve intersection operation by providing extra approach width, allowing more suitable signal

The truck climbing lane (at right) improves capacity and level of service on this 2-lane highway section.

phasing, and preventing the blocking of through traffic by stored vehicles awaiting an opportunity to turn. Separate turning lanes may function under free flow, "yield" or "stop" control, or signal control, as appropriate in each particular case.

Turning lanes often are direct continuations of deceleration lanes. Where storage is required, the storage length should be in addition to that required for deceleration, if a fully adequate length is to be provided.

The reader is referred to Chapter Six for analysis of the effects of separate turning and storage lanes.

AUXILIARY LANES IN WEAVING SECTION

Where two or more traffic flows join and again separate over a relatively short distance with substantial associated weaving between flows, there may be a capacity bottleneck unless additional lanes are provided through the section. This is true both in the case of basic weaving sections where relatively similar flows join, and in the case of weaving sections produced at interchanges by on-ramps followed by off-ramps.

In either case, special capacity analyses are required; simple adjustment factors are not feasible. These procedures are described in Chapters Seven and Eight.

TRUCK CLIMBING LANES AND PASSING BAYS

Although truck climbing lanes and passing bays are clearly types of auxiliary lanes, they are so closely related to the subject of grades that they are included in that section of this chapter.

Surface Condition

A deteriorated, poorly-maintained pavement adversely affects level of service, particularly in terms of speed, comfort, economy, and safety. However, on any highway where capacity is a significant consideration it would be rare for maintenance to be so poor that 30 mph, the approximate speed at which capacity is attained, could not be maintained.

Insufficient data are as yet available to permit development of adjustment factors to

Adverse effect of grades on capacity is alleviated by proper design in this new 2-lane highway section. Note provision of full-width shoulders and over-width earthwork.

reflect the effect of surface condition at other levels of service. It may be assumed that where surface condition is very poor, operating speeds are somewhat lowered for any given volume as compared to those attained where the surface is good. If knowledge of the attainable speed at very low volumes can be obtained on a given road, then an approximate speed-volume relationship can be developed.

Alinement

The alinement and profile of a highway are important factors affecting its traffic-carrying capabilities. Although design speed is a common descriptor of alinement, it is not a sufficient measure for level-of-service purposes, because it takes into consideration only the separate characteristics, such as sharpness of curvature, of individual curves. Thus, it does not vary with the frequency of such curves and the lengths of intervening tangents, although these factors have a pro-

nounced influence on operating speeds. Rather, the lowest design speed of any subsection within the section is often assigned as the governing value for the entire section.

For the relatively long sections of roadway considered for level-of-service purposes, therefore, alinement and profile have been related in this manual to (a) the highway's "average highway speed," and (b) stopping and passing sight distance restrictions.

"Average highway speed," a term originally developed for highway needs study purposes (3), is defined as the weighted average of the design speeds within a highway section, when each subsection within the section is considered to have an individual design speed. It is determined by weighting the design speeds of individual sections of a length of roadway by the length of each section, with suitable allowance for transitions, and is thus a better indicator of the overall influence of alinement limitations on the capabilities of the entire section.

For detailed analyses, determination of average highway speed requires the following information, which reflects the nature of the curves on the route and the extent of the speed changes necessary to negotiate them safely:

1. The geometrics of each horizontal curve and critical vertical curve, including curvature and length.

2. The design speed of each curve (obtainable from AASHO design policies).

3. The approximate distance preceding and following each curve over which speed is affected, together with the average speeds over these deceleration and acceleration distances. (AASHO policies suggest comfortable deceleration and acceleration rates, from which distances can be determined.)

Given the foregoing, a relatively refined speed profile can be obtained for the route, and weighted for development of average highway speed. However, in some regions, for many approximate computations, an "influence area" of 800 ft for each curve has been found to be a workable simplification. Here, specific curve lengths and acceleration and deceleration distances are neglected. Weighting then involves simply considering each curve as an 800-ft length restricted to

the design speed for that particular curve. Although the 800-ft value may not be appropriate everywhere, an equivalent appropriate value might well be found suitable in other areas.

An upper limiting design speed, normally 70 mph, is assigned to tangent sections and sections with easy curvature which satisfy such a limit.

Two types of sight distance requirements are considered in evaluating an alinement —stopping sight distance and passing sight distance. Stopping sight distance is the distance required to bring a vehicle to a stop from a given speed after an object on the roadway becomes visible. Passing sight distance is the minimum sight distance that is required to pass another vehicle safely and comfortably, without affecting the speed of an oncoming vehicle if it comes into view after the passing maneuver is started. For the purposes of this manual this minimum is established as 1,500 ft. Adequate stopping sight distances are necessary continuously on all highways for safety. Passing sight distances require consideration only on two-way roadways with two or three lanes. Although they usually cannot be provided continuously, the more nearly continuous they are the higher the capacity and the better the service provided.

The effect of the quality of alinement on capacity and the service volumes which a roadway can carry, then, is expressed here in terms of the average highway speed and the percentage of the highway having 1,500-ft passing sight distances (for two- or three-lane highways).

These effects are incorporated in the limiting v/c ratios given in the basic computational tables in Chapters Nine and Ten, so do not require independent consideration. They are also shown by means of specific curves in the basic charts there included. For two-lane highways the influences of restricted highway speeds there reported are based on quite detailed studies. For multilane highways less is known and approximations are necessary.

The primary effects apply to levels of service better than capacity. However, capacity itself is somewhat affected. Table 5.3 is presented for information only, to demon-

Multilane freeway retains high capacity level by few changes in alinement and by grade reduction. Note variable median width in adverse terrain.

strate this fact that capacity appears to be at least slightly related to average highway speed even though the operating speeds at capacity are relatively fixed at about 30 mph. It is believed that this apparent effect at

capacity is due mainly to clearance, grade, and related restrictions usually associated with poor alinement, rather than to alinement itself. Hence, no specific adjustment at capacity is included in the computational procedures that follow.

Grades

EFFECTS OF GRADES

Grades affect the capacity of a highway in the following ways:

1. The presence of a grade is generally, although not always, associated with restrictions in the sight distance, thereby affecting the percentage of the length of two-lane highway sections on which passing maneuvers can be performed safely. This effect is considered in the previous section on "Alinement."

2. Vehicle braking distance is less on upgrades and greater on downgrades than on the level, thereby permitting shorter spac-

TABLE 5.3—APPARENT EFFECT OF QUALITY OF ALINEMENT (AS REPRESENTED BY AVERAGE HIGHWAY SPEED) ON CAPACITY

AVERAGE HIGHWAY SPEED (MPH)	CAPACITY (% OF IDEAL ALINEMENT)	
	MULTILANE HIGHWAYS	2-LANE HIGHWAYS
70	100	100
60	100	98
50	96	96
40	—	95
30	—	94

ings between vehicles that are climbing grades, and requiring longer spacings between vehicles descending grades, in order to maintain a safe headway.

3. Trucks with their normal loads travel at slower speeds up grades than on the level,

especially if the upgrade is long and steep. This is also true to some extent for passenger cars. Most passenger cars, however, can negotiate sustained 6 and 7 percent upgrades at speeds above that at which capacity occurs for the highway in question. Therefore,
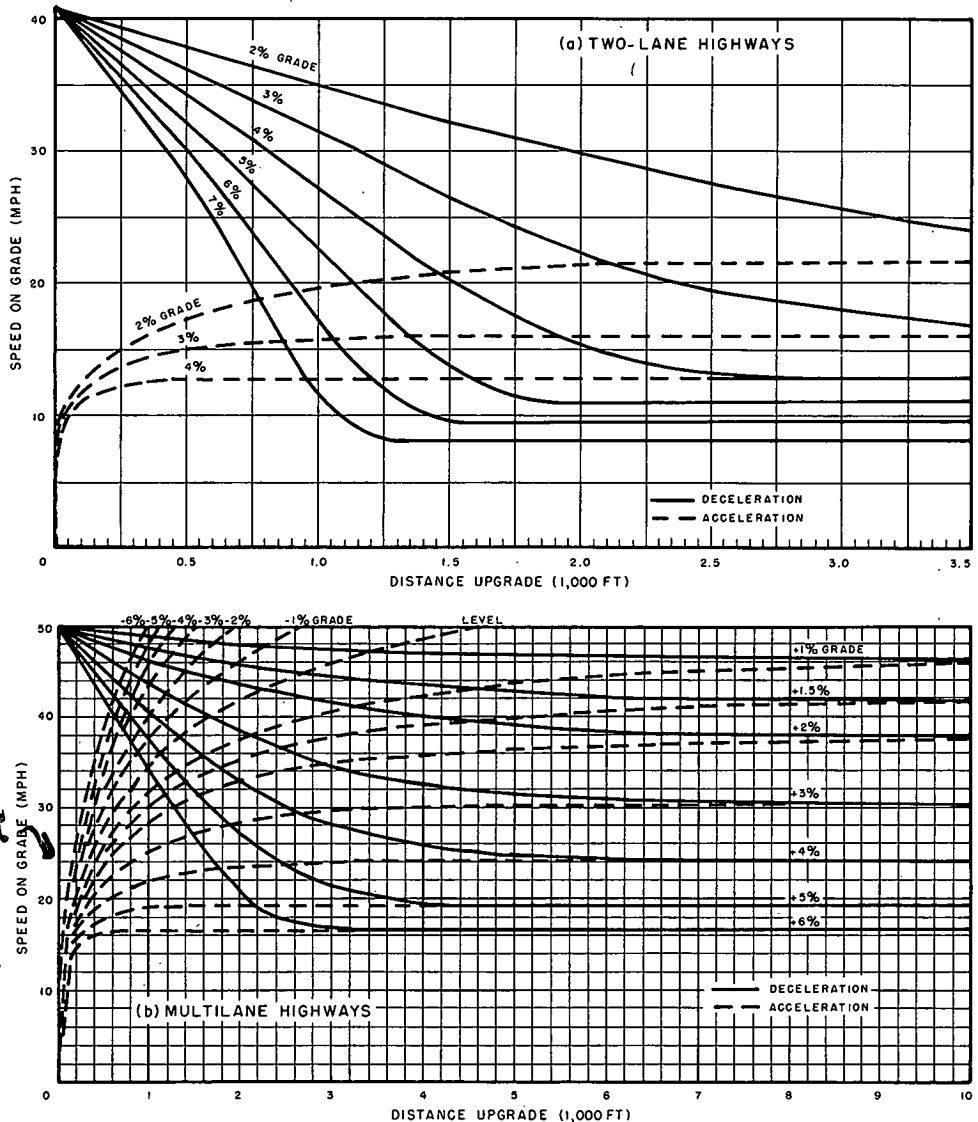
Figure 5.1. Effect of length and steepness of grade on speed of average trucks on (a) two-lane and (b) multilane highways.
(Source: Refs. 3, 4)

the effect of upgrades up to 7 percent on passenger car capacity is generally negligible. It is the effect of sustained steep upgrades on the speeds of trucks, and the resulting effect on capacity, with which this section is concerned.

The relationships between speed of trucks at the bottom of a hill, percentage of grade, and speed at any distance upgrade are shown in Figure 5.1 for two weight/power ratios. Figure 5.1a represents an approximate ratio of 325 lb per hp, considered typical of conditions on two-lane highways carrying a variety of types of trucks (3). Figure 5.1b shows conditions with an approximate ratio of 200 lb per hp, as found on many modern multilane highways carrying largely higher-powered long-haul trucking (4).

From these graphs, for the types of vehicles represented, it is possible to determine how far a vehicle, starting its climb from speeds up to 40 and 50 mph, respectively, can travel up various grades or combinations of grades before the sustained speed is reached. The solid curves indicate the performance that may be expected when the beginning speed is above the possible sustained or crawl speed. They are based on

the assumption that the truck enters the grade at about 50 mph for multilane and about 40 mph for two-lane highways. However, the curves also show the speed reduction due to any length and steepness of grade for other approach speeds. For example, given a typical two-lane condition and a 4 percent grade, if the approach speed is 35 mph (initial distance 400 ft), the speed at a point 1,000 ft up the grade will be 21 mph (final chart distance 1,400 ft).

The broken lines show what performance may be expected when starting on the hill or approaching the hill at a speed lower than the crawl speed, so that the vehicle accelerates to eventually reach the sustained crawl speed. These curves show that long distances are required to accelerate on grades when the approach speed is below the final sustained speed. For example, to change the speed of a typical truck on a two-lane road with a 3 percent grade from 15 mph to the sustained speed of 16 mph, an increase of only 1 mph, the vehicle would have to travel about 900 ft.

Practically any speed reduction by trucks will influence level of service to some degree. Capacity also will always be influenced by trucks to the extent that they take up more roadway space than passenger cars. Nevertheless, the additional influence of grades on capacity will not be felt until they cause truck speeds to fall below 30 mph, the approximate speed at which capacity is generally attained.

As an example, Table 5.4 gives the distance that trucks having a weight-power ratio of 325 lb per hp, considered typical for two-lane highways, can go up various grades before their speeds are reduced to 30 mph, assuming that they enter the grade at 40 mph. It follows that grades longer than those given in the table would have an adverse effect on the capacity of a highway because they would reduce the speeds of trucks that occur with considerable frequency to values below 30 mph.

The distances upgrade in Figure 5.1 are based on uniform grades. Where a vertical curve is part of a length of grade, approximation must be made as to equivalent uniform grade length. Figure 5.2 shows a variety of possible vertical curve configura-

TABLE 5.4—DISTANCE FROM BOTTOM OF GRADE AT WHICH SPEED OF TRUCKS[a] IS REDUCED TO 30 MPH[b]

GRADE (%)	DISTANCE FROM BOTTOM OF GRADE (FT)	VERTICAL CLIMB FROM BOTTOM OF GRADE (FT)
2	1,950	39
3	1,150	35
4	825	33
5	625	31
6	500	30
7	400	28

[a] Trucks having a weight-power ratio of 325 lb per hp.
[b] Assuming an approach speed of 40 mph. Bad alinement, weak or narrow bridges, or other hazardous conditions at the bottom of the hill would make this approach speed unsafe.
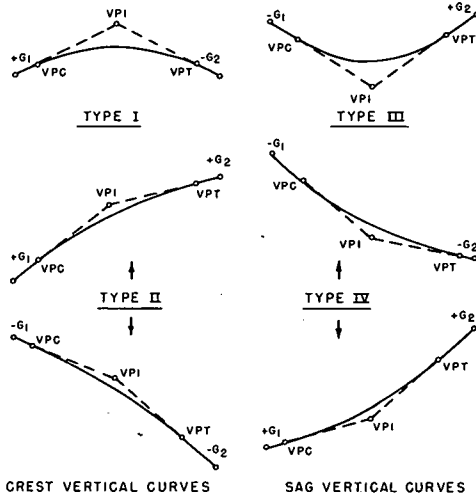
Figure 5.2. Types of vertical curves.
(Source: Ref. *12*)

tions. Where the condition under study involves vertical curves of types II and IV and the algebraic difference in grades is not too great, the measurement of length of grade may be made between the VPI points. Where vertical curves of types I and III are involved, particularly where the algebraic difference in grades is appreciable, about one-quarter of the vertical curve length may be considered as part of the grade under consideration.

Higher weight-horsepower ratios will, likewise, reduce the speed of trucks ascending grades and have an adverse effect on the capacity of a highway. Studies conducted by the Bureau of Public Roads clearly show that weight-horsepower ratios increase with an increase in gross weight (5, 6). Figure 5.3 shows the cumulative frequency distribution of weight-horsepower ratios for all commercial vehicles weighed, both empty
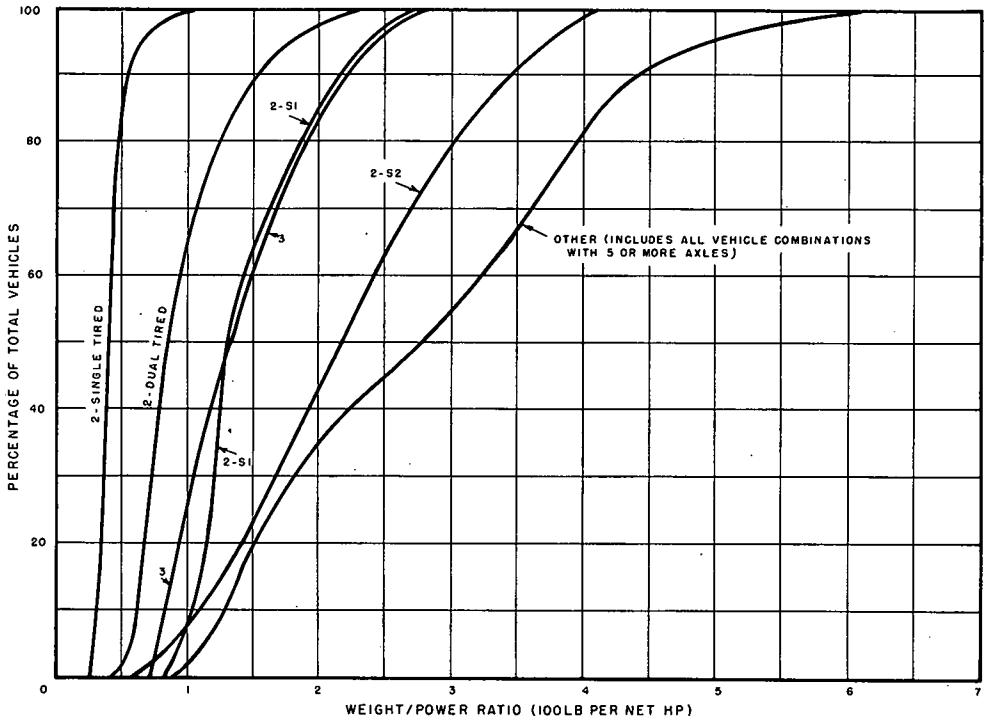
Figure 5.3. Cumulative frequency distributions of weight-power ratios for all commercial vehicles weighed in 1963 studies on major multilane highways.
(Source: Ref. 6)
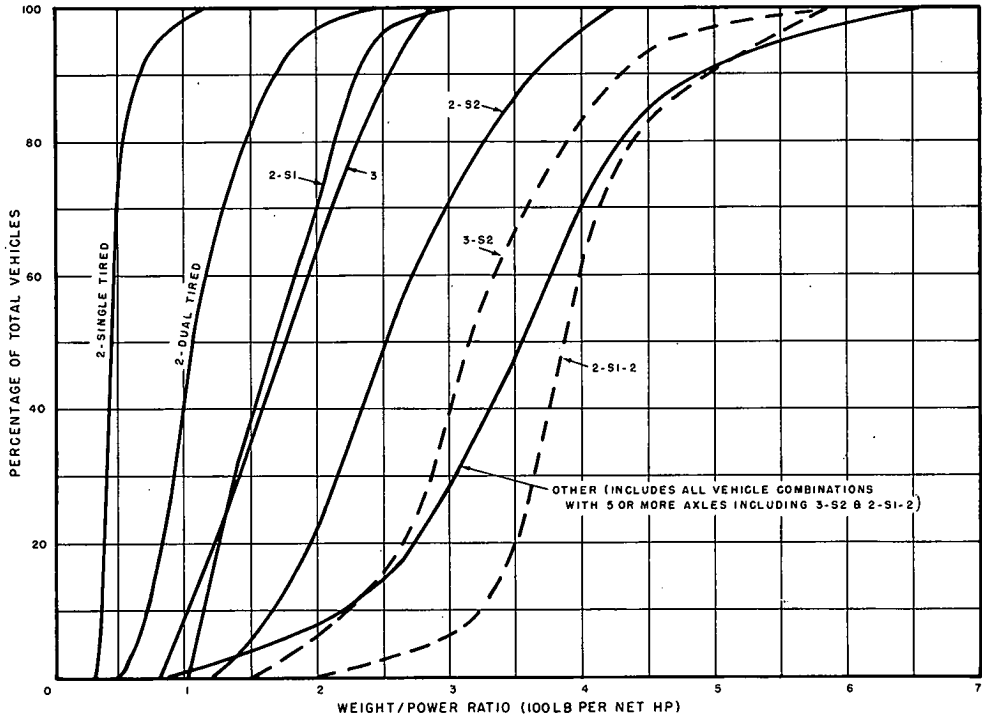
Figure 5.4. Cumulative frequency distributions of ·weight-power ratios for loaded *trucks weighed in 1963 studies on major multilane highways.*

(Source: Ref. 6)

and loaded, during special braking studies conducted in 1963. Figure 5.4 shows similar distributions for loaded trucks only. These curves show that the weight-power ratios of commercial vehicles vary considerably, depending on vehicle type, with a definite increase in weight-power ratios with an increase in the number of axles.

Although engine horsepower has more than tripled during the past 25 years, the overall vehicle performance has not improved as radically. Because increases in horsepower have been offset to a large extent by increases in gross weights, the average weight-horsepower ratio remains about two-thirds of its value 15 years ago.

In typical problem applications it is not the specific speed characteristics at every point on the grade that are directly needed. Rather, the average speed characteristics over grades of various steepnesses · and.

lengths are more useful, where available. Such relationships have been developed for typical two-lane highways (Fig. 5.5). For multilane highways such relationships are more complex and equivalent data are not yet available; alternate approaches to the problem are therefore used.

Knowing ·the effect of a particular grade on the speed of trucks does not in itself enable one to determine its effect on capacity. It is also necessary to know the influence which trucks and buses in the traffic flow have on volume and the effect of each in terms of equivalent passenger cars, or the "passenger car equivalent." Therefore, the information presented in this section is applied in conjunction with that given in the subsequent "Traffic Factors" section, under "Trucks," to determine the overall effects of trucks on grades on the capacity of a given section of highway.
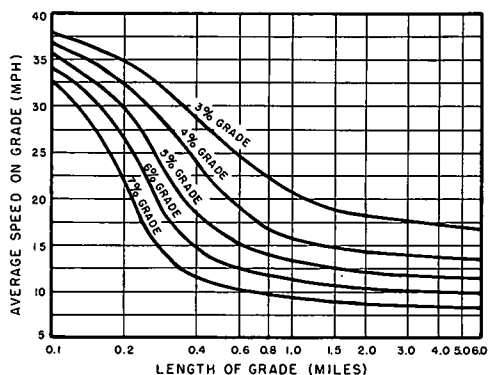
Figure 5.5. Average speed of typical truck over entire length of grade on two-lane highways.

TRUCK CLIMBING LANES AND PASSING BAYS

On long, sustained, steep upgrades the greatest difference occurs between the normal speed of passenger cars and the normal speed of trucks. In effect, trucks on grades take up the space of a larger number of passenger cars (that is, have higher passenger car equivalents) than they do on level sections, particularly on two-lane highways, resulting in lower service volumes and capacities for the uphill sections than for the level sections. Although the need for adequate passing opportunities is therefore greatest on long grades, the passing opportunities, at least on two-lane highways, generally are less here than on level sections.

Truck climbing lanes provide a means for improving both the capacity of and level of service on long, sustained, steep grades carrying significant truck volumes. Under certain conditions, truck climbing lanes will increase the quality of service of an entire two-lane highway section with grades to a level higher than that for the same alinement with no grades. This is because the provision of climbing lanes greatly reduces the effect of trucks on the through lane, while increasing the opportunity for passing. Although actual through-lane volumes may not be greatly reduced, equivalent passenger car volumes are likely to be reduced substantially.

Climbing lanes are also of value in maintaining a balanced level of service along multilane highways, as well as in eliminating potential capacity bottlenecks on such highways. In fact, an added lane for each direction of travel over the entire length on a multilane highway may often be avoided by providing climbing lanes on sustained, steep upgrades. Although, as discussed under "Trucks" in the next section, the effect of trucks on the operation of multilane highways is not as well known as for two-lane highways, it can be generalized that passenger cars tend to shun the right lane whenever slow trucks are in it and volumes are low enough in the remaining lanes so that the desired level of service can be maintained in them. It follows that a climbing lane becomes desirable when without it service would fall below the desired level in the remaining lanes.

Under certain traffic and terrain conditions on exceedingly long grades, the use of passing bays may be an adequate and a more feasible solution than a continuous climbing lane. For example, given moderate traffic volumes and terrain such that widening is not feasible throughout, provision of bays can improve level of service markedly by permitting frequent clearing of queues developing behind trucks, even if capacity is little improved. However, with passing bays the capacity of a highway will generally be greater than without the passing bays, because of the reduced influence of trucks, as discussed in the next section. For certain conditions, they might approach the capacities for highways with a continuous climbing lane. General criteria for their adoption cannot be given; each case must be analyzed separately, because strategic location of the bays is highly important.

In practice then, a climbing lane or passing bay is adopted primarily to maintain a level of traffic operation on the grade in harmony with that elsewhere on the highway. There usually will be little need to make detailed computations of attainable service volumes and capacity of the section with the climbing lane. Rather, the need is to determine these values for the roadway without climbing lanes, to detect where service would fall below the desired level without a climbing lane.

TRAFFIC FACTORS

Highways of identical geometrics (that is, with the same values of all of the roadway factors just described) may nevertheless have differing capacities. This is true because the capacity of a highway is influenced also by the composition and the habits and desires of the traffic which uses it, and by the controls which must be exercised over that traffic. Factors which take these considerations into account are termed traffic factors. Traffic factors considered in this chapter include: trucks, buses, lane distribution, variations in traffic flow, and traffic interruptions.

Trucks

Trucks (defined for capacity purposes as cargo-carrying vehicles with dual tires on one or more axles) reduce the capacity of a highway in terms of total vehicles carried per hour. In effect, each truck displaces several passenger cars in the flow. The number of passenger cars that each dual-tired vehicle represents under specific conditions is termed the "passenger car equivalent" for those conditions. In level terrain where trucks can maintain speeds that equal or approach the speed of passenger cars, it has been found that the average dual-tired vehicle is equivalent, in a capacity sense, to 2 passenger cars on multilane highways and to between 2 and 3 passenger cars on 2-lane highways, depending on the level of service. These values are appropriate for most downgrades as well.

On upgrades, the passenger car equivalent of trucks may vary widely, depending on steepness and length of grade and number of lanes. Further, the average equivalent over a substantial length of roadway will differ from that for specific individual grades within that section.

For approximate analyses of operations on a given highway section it may be sufficient to apply an overall approximate equivalency factor to the route as a whole. In refined analyses, however, truck operations on each of the more significant grades should receive individual attention.

TWO-LANE HIGHWAYS

On two-lane highways, passenger car equivalents of trucks are obtained relatively easily. They can be directly determined by obtaining detailed information on the speeds and headways of vehicles during various rates of flow on highways with different alinements and profiles. An average passenger car equivalent is obtained for the trucks under each condition. If the study is of sufficient magnitude, it is possible to obtain a passenger car equivalent for each type of dual-tired vehicle, classified by speed groups.

Passenger car equivalents can also be calculated with a high degree of accuracy from the separate speed distributions of passenger cars and trucks at any given volume level. The criterion used is the relative number of passings that would be performed per mile of highway if each vehicle continued at its normal speed for the conditions under consideration. That the results from such an analysis agree with those obtained by the more painstaking methods is not surprising. It is the difference between truck speeds and passenger car speeds on grades that causes trucks to reduce the traffic volume carried by a highway at any given level of service. The greater the speed difference, the greater is the reduction in any given service volume, with a corresponding increase in the passenger car equivalents.

On two-lane highways, typical equivalency factors for levels of service B and C, over sections of significant length, including both upgrades and downgrades interspersed with level or crest sections, have been found to average about 5 for rolling terrain and 10 in mountainous terrain. At levels D and E, near capacity, they are 5 and 12, respectively. Table 10.9 gives adjustment factors based on these generalized passenger car equivalency factors, for use in general problems involving such overall roadway sections.

On the other hand, when specific grades on two-lane roads are considered, a wide range in values is found, depending on the severity of the terrain. The passenger car equivalent for any given level of service on sustained steep upgrades increases with a decrease in the truck speed, which is related to the length of grade, because greater

Figure 5.6. Passenger car equivalents for various average truck speeds on two-lane highways.

Figure 5.6 shows how the passenger car equivalent varies with variation in the average speed of trucks climbing any particular grade on a two-lane highway (as shown in Fig. 5.5), for levels of service B, C, and E (capacity). It was developed by the separate speed distributions method just described. It is considered satisfactory in practice to apply the level B criteria to level A also, and the level E criteria to level D.

Reference to Figure 5.6 is not required in most computations, inasmuch as Table 10.10 presents the passenger car equivalency factors for the entire range of grades likely to be found on two-lane roads. These assume average trucks, performing as shown in Figure 5.1a. Where this assumption appears unacceptable and special analyses must be made to determine average truck speed on the grade, Figure 5.6 can be used to determine the equivalency factor. This procedure might also prove necessary in considering a steep downgrade where trucks in low gear travel at a speed slower than passenger car traffic.

Any volume of mixed traffic can be converted to equivalent passenger cars through multiplication by the truck adjustment factor, $(100 - P_T + E_T P_T) / 100$, where P_T is the percentage of trucks and E_T is the appropriate passenger car equivalent determined previously. Similarly, any service volume in passenger cars can be converted to mixed traffic through multiplication by the factor $100 / (100 - P_T + E_T P_T)$. Table 10.10 contains the most used values of this conversion factor.

By relating the equivalent passenger car volume to the capacity of the upgrade section expressed in passenger cars, the effect of the upgrade at any given point may be considered. If the computation indicates that the upgrade in question would experience an unacceptably low level of service, or if capacity would be exceeded, use of truck climbing lanes or passing bays should be considered, as previously discussed.

MULTILANE HIGHWAYS

For multilane highways, truck adjustment procedures are somewhat less well defined, because the quantitative effect of trucks on the capacity of multilane highways with

differences exist between the normal speeds of passenger cars and trucks. It becomes increasingly greater at the poorer levels of service, because passing becomes increasingly difficult, and finally largely impossible. However, on two-lane roads the passenger car equivalent appears to change very little, if at all, with a change in the percentage of trucks in the total traffic stream, for typical truck volumes under constant geometric conditions otherwise. (Studies have not been conducted at locations with more than 20 percent dual-tired trucks and have been confined principally to locations with less than 10 percent of these vehicles during periods of peak flow. It is entirely possible that further studies on two-lane roads might indicate that for certain conditions the passenger car equivalent does change with a change in the percentage of trucks, but as yet there is no evidence to indicate whether it increases or decreases with an increase in the percentage.)

sustained steep grades is not as well known as it is for two-lane highways. The problem is quite complex, involving not only the elements just described for two-lane highways but also such other elements as the distribution of traffic between lanes, truck-passing-truck practices, and the psychological effects of trucks in one lane on drivers in another. These influences make development of passenger car equivalents by either of the methods used for two-lane roads (speed-headway characteristics or passing characteristics) a considerably more formidable task.

It is entirely possible that one or very few heavy trucks on a sustained steep grade of a multilane highway may have nearly as great an effect as a much larger number, at least during relatively low volume conditions, as long as trucks stay in the right lane. Other traffic may largely avoid the right, or truck, lane in either case until volumes in the remaining lane increase to the point where their flows are equally restricted. Where truck-passing-truck maneuvers occur, however, generalizations are not easily made.

Research in this area has been quite limited, and that which has been done has been restricted principally to operation at or near level of service B.* Figure 5.7 shows the results of recent level B research (7). The passenger car equivalents presented in this manual for most capacity and service volume determinations on multilane highways are therefore developed around these limited findings for level B and rationalized values for level E, or capacity, operation (when passing is largely absent), adapted from level B criteria by means of a few field data obtained during capacity operation. Again, 20 percent dual-tired vehicles is the maximum considered, and no distinction is made between freeway and ordinary multilane operation.

On multilane highways, average equivalency factors over relatively long highway sections at levels of service B through E, considering all elements (upgrade, downgrade,

* The Committee considers this area to be one of the most critical voids in existing highway capacity knowledge; further research is urgently needed.

Figure 5.7. Equivalent service volumes relating trucks (vehicles with six or more tires), grades, and passenger cars on two-lane one-way roadways at level of service B.
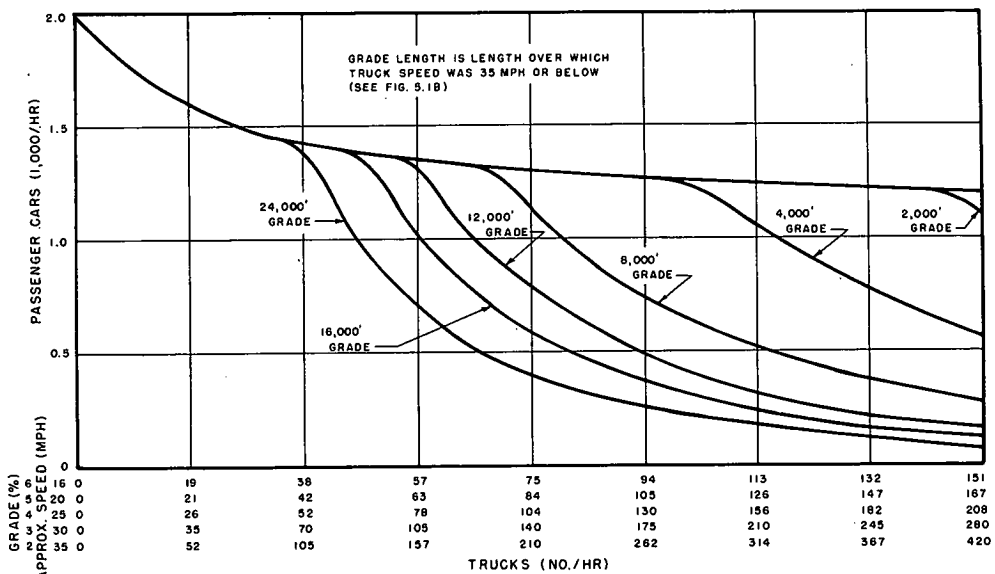
and connecting sections) combined, can be taken as 4 in rolling and 8 in mountainous terrain. Tables 9.3 and 10.3 give adjustment factors based on these passenger car equivalents for use in making overall general computations regarding a long section of multilane highway.

Again, where performance on specific grades is involved a range of equivalency must be considered. Unlike the two-lane case, where little or no passing by trucks is likely at any level of service, considerable passing will occur at the better levels of service unless it is prohibited by local regulation. Therefore, the speed of the slowest truck will not control average truck speeds at such levels, although trucks will influence the second lane as well as the first. At capacity, it is assumed that relatively little passing will occur; hence, the truck influence will be largely limited to the right lane. The percentage of trucks, not considered significant in determining the passenger car equivalent in the two-lane case, must, however, be considered for the multilane case. Although, at capacity, the passenger car equivalent of trucks is nearly as great on multilane as on two-lane highways, given the same conditions (truck performance, length and steepness of grade, and number of trucks in the flow), actually these conditions seldom are the same. Usually, on major highways trucks will be better performing, geometrics will be better, and the trucks may represent a smaller percentage of the total volume. The result is that both the range and absolute values of the equivalency factors used in typical applications are smaller on multilane than on two-lane highways.

Because of the several influencing factors involved, a simple chart comparable to Figure 5.6 for two-lane grades, relating average speed on multilane grades and passenger car equivalent, cannot be developed. However, Tables 9.4 and 10.4 give equivalency factors for the entire range of grades and percentages of trucks likely to be encountered, assuming average performance as shown in Figure 5.1b.

As before, conversion of equivalent passenger car traffic to mixed traffic is accomplished by means of the truck adjust-

TABLE 5.5—AVERAGE SPEED CAPABILITIES OF INTERCITY-TYPE BUSES ON SUSTAINED GRADES [a]

GRADE (%)	SPEED (MPH)
Level	72
+1	68
+2	59
+3	51
+4	44
+5	37
+6	30

[a] Source: Two major bus manufacturers (averaged).

ment factor, $100/(100 - P_T + E_T P_T)$, values of which are given in Tables 9.6 and 10.6.

Buses

Intercity buses in the traffic stream affect capacity in a similar manner to, but to a lesser degree than, trucks. Speed studies have shown that on the average such buses maintain or slightly exceed the average speeds of passenger cars in level or rolling terrain. In mountainous terrain their speeds drop, but remain higher than those of most trucks. Data provided by the major bus manufacturers indicate that typical intercity-type buses can maintain speeds of 30 mph on sustained 6 percent grades. In practice it is seldom feasible to consider buses separately in normal computations, because bus volumes are typically too small to affect the result significantly. Where, however, bus volumes are substantial or heavy grades are encountered, separate consideration can be given to bus operations.

In the absence of more specific data, typical bus speed capabilities are presented in Table 5.5 Equivalency factors based on these speeds are included in Tables 9.3, 9.5, 10.3, 10.5, 10.9, and 10.11.

On two-lane highways the previously-given criteria for trucks can be employed for buses as well, the only change being the use of an appropriate higher average speed for the buses, which results in a lower equivalency factor for otherwise similar conditions.

On multilane highways, particularly free-

ways, recent study results described in Chapter Eleven indicate that a bus may be considered to have an average passenger car equivalent of 1.6 under a wide variety of level and lightly rolling conditions. On heavier grades, little data are available; a factor of 5 for intercity buses is suggested in mountainous terrain, in the absence of better local knowledge. (For heavily-loaded transit buses on the heavy grades found occasionally on suburban freeways in metropolitan areas situated in mountainous terrain, a higher factor may prove necessary, based on local observations. The effects of such local transit buses on urban arterials generally are discussed in Chapter Six.)

A highly-refined procedure would require combined consideration of trucks and buses, but such accuracy is not warranted in most cases within the limits of current knowledge. Separate application of the bus adjustment factor, $(100 - P_B + E_B \, P_B)/100$, to convert mixed traffic to equivalent passenger cars is considered acceptable, where P_B is the percentage of buses in the flow and E_B is the passenger car equivalent of buses. Tables 9.6, 10.6, and 10.12 can be used for buses equally as well as for trucks, to supply this factor for typical cases.

Lane Distribution

On multilane highways all lanes do not carry the same rates of flow (8, 9, 10). For example, on a six-lane freeway section operating at capacity in one direction under ideal conditions at a point away from the influence of interchanges, typical lane volumes might be 1,700 vph in lane 1 (the right lane), 2,100 in lane 2, and 2,200 in lane 3 (adjacent to the median).

No generalized distribution values can be established; a variety of local conditions would have to be considered to establish values for any specific location. For instance, rates up to 2,400 vph might be seen in lane 4 on certain 8-lane freeways. It can be generalized, however, that for ideal conditions, greater use is made of the left lanes and less use of lane 1 than would be indicated by the average lane volume at each level of service.

The left lanes, under ideal conditions, carry higher volumes partly because the fastest drivers tend to avoid lane 1, in which most of the slower drivers remain. Many drivers in the left lanes also are avoiding the "turbulence" in lane 1 caused by the effects of entrances and exits, most of which are on the right on freeways.

On freeways having more than two lanes in one direction of travel, however, certain generalizations are possible regarding the relative efficiency of various numbers of lanes. More efficient use usually is made of the additional lane(s) above two than the average lane volume for two lanes in one direction. This is because the likelihood of becoming "trapped" behind slower moving vehicles is greatly reduced with more than two lanes in one direction of travel, given the same average rate of flow per lane. The degree of this increase in efficiency is reasonably predictable. For instance, the total rate of flow for two lanes in one direction at level of service B is 2,000 pcph, or an average of 1,000 pcph per lane, but addition of a third lane increases the total flow to 3,500 pcph, on the average, for the same level of service. The added third lane thus, in effect, adds 1,500 pcph, instead of 1,000 pcph, to the service volume of the one direction for level of service B, resulting in an average flow of 1,167 pcph per lane. At lower levels of service with higher rates of flow this effect diminishes, becoming negligible at capacity (level of service E). Chapter Nine presents these data in detail. It should be noted that, still, only total volumes and average-per-lane volumes are known; lane distributions remain variable.

Lane distributions at critical areas of operation along a highway may vary considerably from those likely under ideal conditions. In design of high-type multilane highways, particularly freeways, or in evaluation of their operation, knowledge of how traffic redistributes itself at critical areas is essential.

For ramp entrances and exits, and weaving areas between adjacent ramp terminals, the volume in lane 1 is especially significant, because it becomes the measure of how much traffic may enter or leave the highway under acceptable operating conditions. In the case of weaving areas involving major traffic

movements, sufficient width must be pro-
vided to accommodate both the weaving
traffic and the non-weaving flows on either
side. On upgrades distributions must
change because, although most of the trucks
stay in lane 1, their passenger car equiva-
lents increase markedly, and in effect they
take over space previously occupied by ac-
tual passenger cars, which move into other
lanes.

Service volumes at any given speed, and
usually capacity itself, are reduced on multi-
lane highways with no or only partial con-
trol of access, where vehicles regularly en-
ter and leave on both the right and left
along the highway. On such highways, the
resulting slower speeds and turbulence in
the left lane will make it more comparable
to lane 1, where slow vehicles and turbu-
lence will continue to prevail, and any lane
between these outer lanes will carry a
greater proportion of the traffic.

Although lane distribution is an important
variable, no special adjustment for it need
be made because, where its significance is
great enough to warrant consideration, as on
certain freeways, grades, or at ramp junc-
tions, its effect is already taken into account
in the basic procedures employed.

Variations in Traffic Flow

As discussed in Chapter Three, the de-
sign hourly volume is determined as a per-
centage of the assigned future ADT volume.
This procedure reasonably accounts for the
variation in traffic volumes during the differ-
ent hours of the day, and even the fluctua-
tions in hourly volumes throughout the year.
As also discussed in Chapter Three, varia-
tions in flow within the peak hour also have
definite effects on the operating characteris-
tics of a highway, and thus influence the
capacity which can be attained in practice.
Knowledge of these effects is increasing, at
least for certain highway types.

In this manual the peaking characteristics
of traffic on freeways and through at-grade
intersections are taken into account. They
are expressed in terms of the peak-hour fac-
tor, which is the ratio of the volume occur-
ring during the peak hour to the maximum
rate of flow during a given time period
within the peak hour. For freeways this

ratio is based on the maximum 5-min rate
of flow within the peak hour, whereas for
intersections the maximum 15-min rate of
flow is used. These ratios are determined by
dividing the number of vehicles actually
passing during the peak hour by, respec-
tively, twelve times the number of vehicles
passing during the peak 5-min period or
four times the number passing in the peak
15 min. The maximum attainable value of
the peak-hour factor is 1.0.

Studies have shown that the highest 5-
min rate of flow within the peak hour on
urban freeways is usually 1.05 to 1.15 times
the peak-hour volume in larger metropoli-
tan areas; it may range up to about 1.4
times the peak-hour volume in smaller met-
ropolitan areas. This range is equivalent to
a peak-hour factor ranging from 0.95 to
0.70. Likewise, through a typical at-grade
intersection the highest 15-min rate of flow
may be in the order of about 1.2 times the
peak-hour volume, giving a peak-hour fac-
tor in the neighborhood of 0.85.

Peak-hour factors should be applied in
computing capacities and service volumes of
at-grade intersections and freeways. Their
selection and use is described in detail in
Chapters Six and Nine, respectively. Less
is known about the peaking characteristics
in uninterrupted flows on sections of rural
highways, particularly those without con-
trol of access. However, since these are
normally designed for a relatively high level
of service, within the particular class of
highway, a large safety factor with regard
to peaking is usually provided. Therefore,
variations in traffic flow within the peak
hour are not normally considered in their
design. Similarly, little is known about peak-
ing characteristics on ordinary urban and
suburban highways with relatively uninter-
rupted flow, and without access control, but
again the need is not great. Uninterrupted
urban flow seldom is found; at-grade inter-
sections or other interrupting features along
urban and suburban routes usually consti-
tute the critical areas with respect to ca-
pacity.

Traffic Interruptions

Thus far discussion has centered princi-
pally about uninterrupted flow. Obviously,

Proper fitting of this 4-lane highway into difficult terrain reduced severe alinement effects that could greatly reduce capacity.

where features are built into the highway which force some or all traffic to stop, the highway's ability to carry traffic will be impeded. Although level of service will suffer to a greater or lesser degree in every case, typical occasional interruptions will accommodate, with only momentary restriction, all traffic flowing at the better levels of service on the class of highway involved. However, the poorer levels of service and capacity will suffer unless sufficient additional traffic lanes are provided through the vicinity of the restriction to offset the reduced time utilization of the normal number of lanes.

As previously mentioned, a basic rule to remember in considering a traffic interruption is that consecutive passenger vehicles stopped in line will rarely get under way at a faster rate, on the average, than 1,500 pcph per lane (average of 2.4 sec headway). When it is recalled that uninterrupted flows may carry some 2,000 pcph per lane, it is easy to see why back-ups can develop rapidly where traffic is stopped, unless appropriate steps are taken.

For the purpose of this discussion, traffic interruptions are divided into two broad categories—at-grade intersections and other interruptions.

AT-GRADE INTERSECTIONS

At-grade intersections constitute by far the most common type of interruption, and the most difficult to eliminate, because they involve the sharing of a common area of roadway by two or more entirely different traffic flows. Their influences on capacity and service volumes are so great, in most cases where they exist, that they govern the capacity determination, rather than being handled as adjustments to uninterrupted flow criteria. An entire chapter of this manual (Chapter Six) is devoted to this subject.

OTHER INTERRUPTIONS

This category includes such interruptions as toll gates, drawbridges, and at-grade railroad crossings. At toll gates all vehicles must stop, but generally there is not excessive delay and queuing because adequate additional lanes normally are provided to assure that demand does not exceed total gate capacity. Elsewhere, however, such compensating elements are not so easily provided. No general adjustment factors can be provided to correct for such influences. Each is a special case, which must be considered individually. Once the magnitude of the problem is established, feasible corrective measures can be considered.

For example, suppose that an at-grade railroad crossing of a four-lane highway were closed for 2 min while the rate of flow per lane in the direction of heavier travel was 600 pcph, or 10 per min. Some 20 cars would be backed up in each lane when the crossing reopened and flow resumed, with the queues extending back for a distance of 500 ft, if the average distance per stored vehicle were 25 ft. Because the front of the queue would resume travel at the rate of 1,500 pcph per lane, whereas vehicles at the back of the queue would be arriving and stopping at the rate of only 600 pcph per lane, the queue would soon dissipate, but would move back upstream as it did so.

If, however, at the same location, traffic in the direction of heavier travel were flowing at the rate of 1,500 pcph per lane or higher, the queue would be correspondingly longer at the end of the 2-min stoppage. More important, the queue would not dissipate as it moved back upstream until the rate of flow of arriving vehicles fell below 1,500 pcph per lane, because until this happened traffic would be arriving at the rear of the queue at the same or at a greater rate than it was pulling away from the front. The queue, or stoppage wave, might well move back several miles before reaching a time or place where demand fell below 1,500 pcph per lane.

The type of operation described for the railroad crossing, however, does not occur at an overloaded toll gate. Where demand is sufficiently great to cause queues to form at a toll gate, each vehicle does not remain stopped in one place for some period of time and then resume normal travel. Instead, it moves up to the gate in a "stop-go" pattern of operation.

If a queue sufficiently long to keep a continuous backlog of traffic in spite of random arrival were maintained at each lane of the gate, the capacity of each lane would be controlled by the average time required for each vehicle to pass through (average headway through the gate). However, queues of such length generally would be intolerable to the highway users. To prevent formation of queues of excessive length, the determination of the number of lanes at a toll gate or other similar interruption should be based on queuing theory, which considers the effect of random arrival, as discussed in specialized statistics texts.

Actually, to a varying degree even speed limits may be traffic interruptions. If they reduce speeds only to something higher than approximately 30 mph (or possibly 40 mph on freeways) they do not greatly influence capacity, but they do affect level of service. On the other hand, a rigidly-enforced limit of 25 mph or less, on a highway with uninterrupted flow otherwise, will reduce capacity by preventing attainment of the optimum speed for capacity.

APPLICATION OF ADJUSTMENT PROCEDURES

Capacities under ideal conditions for various highway types were discussed in Chapter Four. In practice, these values for ideal conditions are never directly applicable to a specific roadway, except possibly certain high-type parkways which carry no trucks.

In order to solve the usual problem of estimating the capacity of any given individual roadway, the values obtained from Table 4.1 must be adjusted downward to take into account the several roadway and traffic factors which have been described in this chapter.

Specific adjustment procedures for these factors are presented where appropriate in the following chapters. At this point, it will suffice to list the several factors that have been discussed, to indicate which must be considered in typical problem solutions, and to show how each of the latter is considered. This listing is given in Table 5.6.

The factors included in the several tables in this chapter, and summarized in the foregoing, are applicable only under uninterrupted flow conditions, except as specifically stated otherwise in the case of several which relate to intersection operation. The remainder are not applicable for city streets or other facilities where average operating speeds are low and stop-and-go operation

TABLE 5.6—SUMMARY OF ADJUSTMENTS TO IDEAL UNINTERRUPTED FLOW VALUES [a]

FACTOR	HANDLING
Roadway factors:	
Lane width ⎫ Lateral clearance ⎭	Select appropriate adjustment factor from Tables 9.2, 10.2, or 10.8.
Shoulders	Apply adjustment factor as given under "Shoulders" if applicable.
Auxiliary lanes	
Parking lanes	Consider as lateral clearance problem.
Speed change lanes	Consider as ramp junction problem; see Chapter Eight.
Turning and storage lanes	Normally not associated with uninterrupted flow; see Chapter Six for handling of signalized intersections.
Weaving lanes	Consider as weaving or ramp junction problem; see Chapters Seven or Eight, as appropriate.
Truck climbing lanes	Consider in conjunction with "Trucks," below.
Surface condition	No specific adjustment; judgment required.
Alinement [b]	Built into other adjustments, mainly v/c ratio.
Grades	Combined into "Trucks" adjustment, below.
Traffic factors:	
Trucks, Two-lane	Apply equivalent passenger car procedures (see Chapter Ten, including Tables 10.9, 10.10, and 10.12).
Multilane	Apply equivalent passenger car procedures (see Chapters Nine and Ten, including Tables 9.3, 9.4, 9.6, 10.3, 10.4, and 10.6).
Buses	Apply equivalent passenger car procedures (see Chapters Nine and Ten, including Tables 9.5, 9.6, 10.5, 10.6, 10.11, and 10.12).
Lane distribution	Built into other adjustments.
Variations in traffic flow	For freeways, apply peak-hour factor (5-min base); see Chapter Nine.
	For intersections, apply peak-hour factor (15-min base); see Chapter Six.
	Optional, based on judgment, for other highway types.
Traffic interruptions	Intersections; see Chapter Six.
	Other; each a special case.

[a] As presented in Table 4.1.
[b] Including average highway speed and passing sight distance.
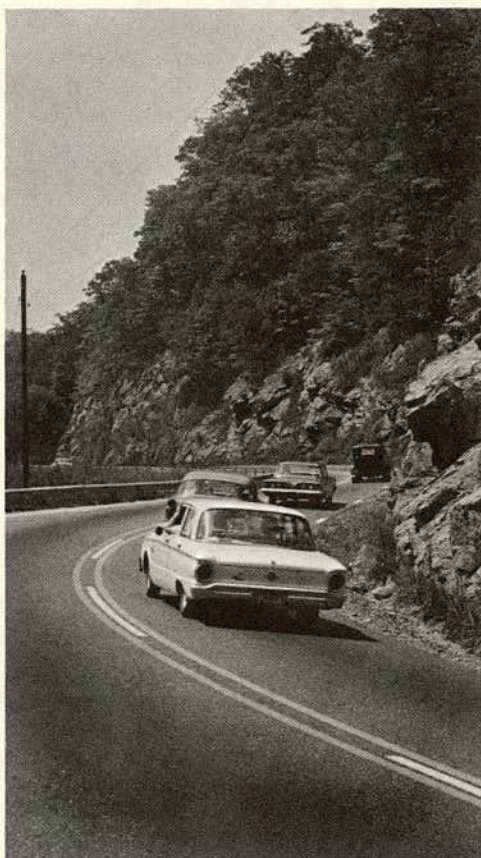
Lack of auxiliary climbing lane, adequate shoulders, and proper alinement reduce capacity of this 2-lane highway.

prevails. Specific procedures for such highways are discussed where appropriate in Chapters Six and Ten. It should also be noted that these tables show only the effect of restrictive elements on capacity and level of service, as measured by the criteria selected in Chapter Four. The adverse effect on driver comfort and safety is not indicated, but exists at all levels of service.

It is important that in actual applications good judgment be used in considering the factors discussed throughout this chapter. Any final problem solution certainly should reflect full consideration of each factor. But

it might be unnecessarily time consuming to consider each in detail during preliminary computations. This would be particularly true where comparative analyses were being made, requiring relative rather than absolute answers. In short, the refinement of capacity and service volume computations, including application of adjustments, should be commensurate with the degree of accuracy feasible for the problem at hand.

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CHAPTER SIX

AT-GRADE INTERSECTIONS

One of the more important elements limiting, and often interrupting, the flow of traffic on a highway, especially one in an urban area, is the intersection at grade. Intersections not only control, to a large extent, the capability of major and secondary arterial streets to accommodate the flow of vehicles and pedestrians, but they also may seriously affect or limit the ability of nearby freeways to perform at maximum efficiency. Therefore, the subjects of intersection operation and interrupted flow are often largely synonymous.

The amount of vehicular traffic which can approach and pass through an intersection depends on (a) various physical and operating characteristics of the roadways, (b) environmental conditions which have a bearing on the experience and actions of the driver, (c) characteristics of the traffic stream, and (d) traffic control measures.

Because so many such factors influence interrupted flow through intersections, it is not feasible to define "ideal conditions" as was done in the case of uninterrupted flow. Consequently, capacities and service volumes under ideal conditions cannot be directly specified. Rather, interrupted flow criteria are developed around typical or average conditions. Adjustments either upward or downward may be applied to fit the specific conditions at hand.

Although the volume of vehicular traffic which actually can reach and pass through an intersection may well be influenced by conditions remote from the intersection, the capacity of any specific intersection is determined largely by the effect of elements directly related to its contiguous approach roadways. Seldom are all approaches to an intersection simultaneously burdened to their full capabilities. Therefore, it is appropriate that intersection capacity be thought of in terms of the separate capacities of each individual approach roadway. In practice, the term "intersection capacity" as generally used by traffic engineers actually represents individual approach capacity. Similarly, "intersection service volume" in practice usually refers to the service volume on a particular approach. In this connection, it should be noted that the term "intersection capacity" often will be broadly used for simplicity in referring to the entire field of capacities and service volumes.

This chapter is primarily concerned with signalized intersections, but a brief discussion of unsignalized intersection operations is included.

SIGNALIZED INTERSECTIONS—GENERAL

In the preparation of the 1950 Highway Capacity Manual, the Highway Capacity Committee found that little material had been published on the subject of signalized intersection capacity. Therefore, under the direction of the late O. K. Normann, considerable research was performed to assemble satisfactory data. In addition to the data collected by committee members, material was furnished by state highway departments and by officials of many cities, as a result of solicitation by the Bureau of Public Roads. Through such splendid cooperation, information was obtained on a scale never before attempted. From these data, which reported maximum observed volumes per hour recorded in 15-min increments, intersection capacity curves and associated adjustment factors were derived which served to meet a very pressing national need.

During the following years, however, it was recognized that still more refined data would be required to detect and explain the effect of additional variables involved in

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signalized intersection capacity. In 1954, a new, more comprehensive study was inaugurated by the Highway Capacity Committee, again under Mr. Normann's personal direction. Special forms and instructions were sent to responsible officials throughout the country with the request that they be completed and returned to the Bureau of Public Roads for analysis. Detailed information was obtained during 1955 and 1956 from some 1,600 busy intersection approaches, recorded on a per-cycle basis for from 1 to 2 hr, with the degree of utilization noted for each cycle. Data from about 1,100 of these approaches were selected as suitable for detailed analysis. These data were analyzed both graphically and by multiple regression techniques (1). Most of the final analyses, interpretations of results, and preparation of basic computational charts and procedures were carried out personally by Mr. Normann in the period immediately prior to his untimely death (2). It is, therefore, fitting that the procedures described in this chapter be designated collectively as the "Normann method" of computing signalized intersection capacity and levels of service.

Despite the knowledge thus gained, there remain a number of factors that influence intersection capacity and service volumes for which only limited data are available. Particularly perplexing among these is the effect of environmental and operating characteristics which vary widely between different localities, and which may considerably affect the number of vehicles that can pass through an intersection in a given period of time. Local traffic regulations, degree of enforcement, and education and training of drivers are among the several elements that comprise these environmental conditions which cannot be precisely evaluated from material at hand. Thus, although the effects of these elements are inherent parts of the reported data, they cannot be identified as yet; they undoubtedly account for much of the remaining variability between predicted and actual capacities and service volumes.

Basically, a signalized intersection consists of the actual intersection area (that is, the section of pavement shared by traffic on each of two crossing streets) and a num-

TABLE 6.1—FACTORS AFFECTING INTERSECTION CAPACITY AND LEVELS OF SERVICE

Physical and operating conditions:
    Width of approach
    One-way or two-way operation
    Parking conditions
Environmental conditions:
    Load factor
    Peak-hour factor
    Metropolitan area population
    Location within metropolitan area
Traffic characteristics:
    Turning movements
    Trucks and through buses
    Local transit buses
Control measures:
    Traffic signals
    Marking of approach lanes

ber of roadways on which vehicles approach and leave the intersection area. Traffic signals and other control devices (i.e., signs, lane markings, turning controls, etc.) are used to specify the conditions—direction, sequence, and time—governing movement of traffic.

The number of vehicles that can enter an intersection is dependent on a large number of factors, some of which are fixed or semifixed (such as basic geometric design and built-in traffic control features), and others of which are variable (reflecting the actual use of the intersection by vehicles and pedestrians). Such factors must be considered if a computed capacity or service volume value is to be meaningful. The factors discussed in this chapter are summarized in Table 6.1.

FACTORS AFFECTING SIGNALIZED INTERSECTION CAPACITY

Basic Physical and Operating Conditions

The geometrics and dimensions of the streets involved, the proportion of the total pavement width available to moving traffic, and the manner in which traffic is handled on that pavement are all fundamental factors in-

Principal street in central business district providing six traffic lanes and two parking lanes. Note traffic making left-turn maneuver in foreground on advance signal control before opposing traffic starts.

fluencing the traffic-carrying capabilities of intersections along those streets. Therefore, approach width in feet, parking conditions, and type of operation (one-way or two-way) are used in this procedure to establish a basic condition under which the other factors can be evaluated.

WIDTH OF APPROACH

The width of the approach, rather than the number of traffic lanes, has proved to have the most significant bearing on the capacity of a typical approach. Obviously, approach widths vary greatly at intersections, depending on the street width and the positioning of pavement markings and other traffic control devices. Sometimes widths are deliberately varied within the day to handle normal use variations through the day,

occasionally by means of complex lane control signal installations, but often by means of movable traffic cones or only simple signing.

The number of lanes actually formed by moving traffic thus may change, either due to regulation or simply because striped lanes are not always respected during peak hours, where physical room exists for more lanes than have been marked. Available data indicate that, fundamentally but with reasonable tolerance, both intersection approach capacity and service volumes vary directly with the width of approach. Therefore, the procedures that follow are based on the widths of approach roadway, rather than on the number of lanes.

This is not to say, however, that striped lanes have no effect on intersection capacity. Where inefficient use of an approach is

occurring without striping, marked lanes may prove very beneficial. This subject is further discussed in the section on traffic control measures.

One caution is necessary here. No longer is it feasible to consider the approach width to be simply one-half of the curb-to-curb width, as was done in earlier procedures. Offset center lines are very common today, making it essential that the actual width available for approach traffic be known and utilized.

PARKING CONDITIONS

Parking regulations on an approach might well be considered to be traffic control measures, in terms of their usual handling by signing and police enforcement. However, because parking conditions at or near an approach have such a pronounced effect on intersection capacity, the presence or absence of parking is considered to be a basic condition which should be defined initially before other factors are evaluated. The removal of parking provides a substantial gain in capacity. If the elimination (or addition) of parking is being considered on one or both sides of an approach, capacity should be evaluated for each condition.

In this connection it is important to repeat a previously-mentioned point—namely, that the width of roadway influenced by a parked vehicle is, on the average, substantially greater than simply the physical space it occupies. The cautious reactions of passing drivers, who fear sudden maneuvers by parked vehicles or doors opening into their paths, result in effective loss of some 12 to 14 ft of roadway width, on the average. Where a wide approach exists and load factors are low, this lateral shying away may produce an effective width loss due to parking ranging up to 20 ft or more. At the other extreme, where the approach is narrow and loading is so high that there is little or no room for maneuvering, the effective width loss may be little more than that taken up physically by the parked cars. Trucks, of course, take up greater physical space.

"No parking" has been defined as no standing and no stopping on the approach, other than occasional passenger discharge or pickup. "With parking" means that vehicles are present, standing attended or unattended, along the curb on the approach. For capacity purposes, it is the actual presence or absence of parked vehicles, not posted parking regulations, which counts.

When "no parking" conditions are specified, this does not necessarily mean that no parking can exist anywhere in the block; rather it indicates that parking must not exist close enough to the intersection to affect the approach's operation. Attempts to develop refined criteria regarding the influence of this parking restriction distance, using the data from the 1955-1956 studies, have proved largely unsuccessful.

As a rule of thumb, approaches having parking within 250 ft of the intersection should be considered in the "with parking" group. However, many exceptions to this rule exist. For example, on a street lacking coordinated signalization and with a relatively small percentage of green signal time, the midblock section may well be able to accommodate parking quite close to the approach while still handling sufficient moving traffic to make full use of the limited green time at the approach. At the other extreme, on a street with a perfectly-coordinated signal system, no midblock parking can be tolerated in the "no parking" condition because uniform width throughout is required to handle moving platoons of traffic.

ONE-WAY OR TWO-WAY OPERATION

There are obviously major differences in the operation of one-way and two-way approaches which are reflected in the capacities and service volumes attained. On a one-way approach, for example, left-turning movements can be made more easily, due to the absence of opposing traffic. Where cross streets are also one-way, turning movement conflicts are further reduced. In either case, the reduction in total possible movements reduces pedestrian-vehicular conflicts. The one-way street also lends itself to better signal progression.

Because of these differences, capacity analysis procedures for one-way and two-way approaches are handled separately in

Well-marked pavement and prohibition of parking expedite traffic at this pair of intersections in an outlying business area.

this manual, and differing adjustment factors for these two conditions are presented.

In most cases, these procedures show that a given approach width will have somewhat greater capacity operating one-way than two-way, due to the reduced friction. However, this will not always be the case. Consequently, it is not prudent to make broad generalizations regarding the relative efficiency of an individual approach operating one-way as compared to two-way, without consideration of the entire system of which the approach is a part. In particular, comparisons made by superimposing the one-way curves on the two-way should be avoided. This subject of relative efficiency is discussed in detail in Chapter Ten, in the section on urban arterials.

Environmental Conditions

Environmental condition factors represent those characteristics of the traffic demand, as reflected in the traffic stream, that cannot be readily changed by alteration of design or control features of the intersection. These factors include load factor, peak-hour

factor, metropolitan area population, and location within a city.

All of these factors except the last are additions not included in earlier procedures. In effect, the new factors are, together, a formalized substitute for the "city factors" which individual traffic engineers frequently found need to apply to the earlier average curves. They help to explain why such city factors were found necessary in the past.

LOAD FACTOR

The term "loaded cycle," or more properly "loaded phase," is much used in describing the degree of utilization of an individual intersection approach. A green phase on an approach may be considered to be "loaded" when the following conditions apply: (1) there are vehicles ready to enter the intersection in all lanes when the signal turns green, and (2) they continue to be available to enter in all lanes during the entire phase with no unused time or exceedingly long spacings between vehicles at any time due to lack of traffic, whether resulting

from lack of demand (as often found toward the end of the phase) or from frictional interferences upstream. Usually, the ending of a loaded green phase will force some vehicles to stop, but in a perfect progressive signal system the last of a platoon of vehicles may pass through just as the signal changes.

It is not essential for traffic to *move* continuously during a phase for that phase to be considered loaded; all that is required is that the vehicles be present, and that any stoppages which occur be due to conditions at the intersection under study, not a consequence of conditions elsewhere. For instance, even if the left lane did not move at all during a green phase, because the lead car desired to turn left but could not do so due to opposing traffic, the phase could still be considered loaded if all other criteria for loading were met. On the other hand, if traffic could not move because of a back-up from an intersection ahead the location under study would not be loaded. Rather, it would be "jammed" and flow data from it would be quite meaningless.

The load factor is a measure of this degree of utilization of an intersection approach roadway during one hour of peak traffic flow. It is the ratio of the number of green phases that are loaded, or fully utilized, by traffic (usually during the peak hour) to the total number of green phases available for that approach during the same period. As such it is also a measure of the level of service on the approach, as discussed later. The load factor for a normal intersection may range from a value of 0.0 to a value of 1.0.

A load factor of 0.0 represents any situation in which no cycle during the hour is loaded. Hence, it represents a wide range of excellent or very good operating conditions, handling volumes from very few vehicles up to the point where most green phases are almost fully utilized.

A load factor of 0.2 still indicates a good operating condition for most cases. It represents a condition where some 20 percent of the phases are fully utilized but the remaining cycles are operating below this level.

A load factor of 0.4 represents a relatively high volume condition which may result in considerable delay to some vehicles on the approach. Load factors greater than 0.4 represent correspondingly higher percentages of fully utilized phases. These conditions are not often experienced at isolated non-interconnected intersections. However, a progressive system will accommodate high volumes with a high load factor, provided it is accurately operated and timed for traffic demand. Nevertheless, it is rare for a load factor of exactly 1.00 to be attained, even under the best operating conditions under high volumes, due to normal fluctuations in traffic flow. Where a load factor of 1.00 was reported during the 1955-1956 studies, it was frequently discovered that a "jammed" condition due to congestion somewhere else downstream had been erroneously interpreted as full utilization.

Figure 6.1 shows typical relationships between load factor and delay likely to be encountered by traffic, for the simple case of a single-lane approach at an isolated intersection. Figure 6.1a shows the typical demand fluctuations which tend to prevent attainment of a load factor of 1.00 even under overall high-volume conditions when demand exceeds capacity, on the average over an extended period, and long back-ups of traffic are resulting. Even under these highly undesirable conditions, when many vehicles are delayed through several signal cycles before clearing the intersection, there are often short periods within a full hour when conditions are less critical. In the case shown, for example, all waiting vehicles cleared in two of the cycles early in the hour, while demand was building up.

Figure 6.1b shows the result of a moderate (20 percent) reduction in demand. Over one-half of the cycles are still loaded (load factor = 0.57), but few vehicles must wait through more than one signal cycle.

It is important to stress that load factor as discussed so far applies to a single approach of a given intersection. Obviously, insofar as the individual approach is concerned the load factor can be readily altered simply by changing the signal timing split. In practice, however, no one approach can be considered alone. Rather, signal operation must be provided which will properly balance the needs of both streets involved. This does
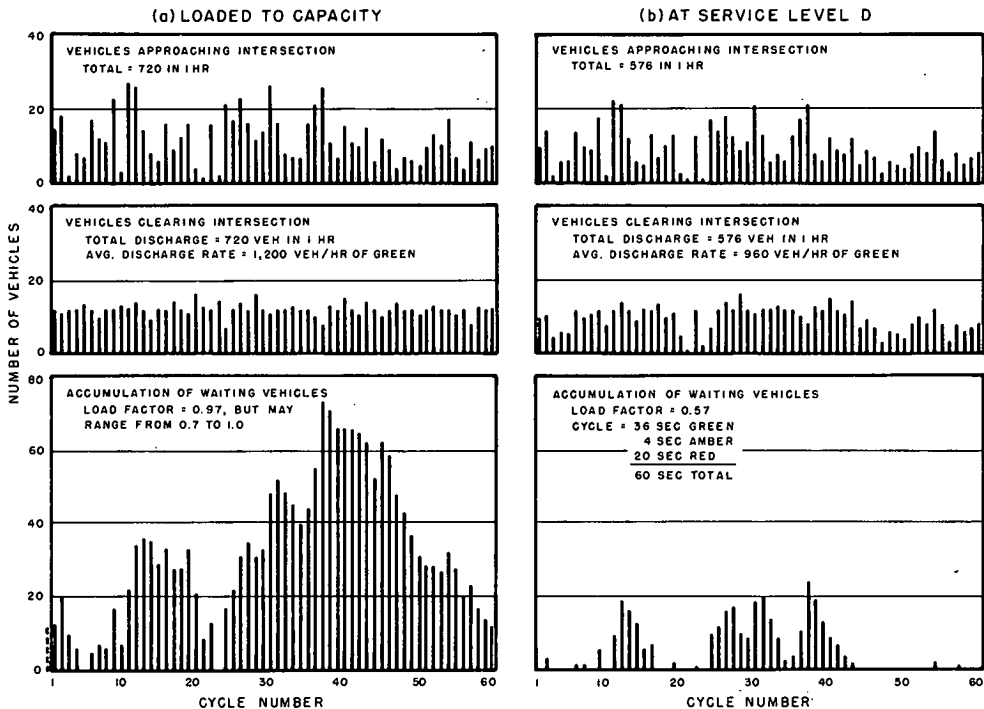
Figure 6.1. Operation of traffic at a rural intersection (a) loaded to capacity and (b) in intersection service level D.

not necessarily mean that the same level of service must be provided on both. Where two arterials cross equal levels probably would be desired, but where the side street is decidedly inferior to the arterial differing levels might be appropriate.

Further considerations relating to load factor and its use are included in the portions of this chapter relating to levels of service and to intersection analysis procedures.

PEAK-HOUR FACTOR

The subject of individual approach loading is closely related to the consideration of overall delays to traffic along a route and, thus, to travel times. Small reductions in flow rate often substantially reduce travel times by largely eliminating back-ups. This is of major importance at locations where there is a large variation in the demand on the individual streets during the peak hour

as well as during different hours of the day.

In modern traffic engineering applications it is often not enough to know only that adequate capacity exists to handle, in one hour, the total traffic that will arrive in that hour. Variations in the demand throughout the hour may produce peak arrival rates for short periods during the hour which substantially exceed the average rate. This is the situation represented by Figure 6.1a. To assure that long back-ups do not develop during parts of an hour, even though capacity for the hour is not exceeded, this element must be taken into consideration. The peak 15-min flow is used as the basic short-period rate at intersections for such consideration in this manual.

The peak-hour factor is a measure of consistency of demand. For intersections it is defined as the ratio between the number of vehicles counted during the peak hour and four times the number of vehicles
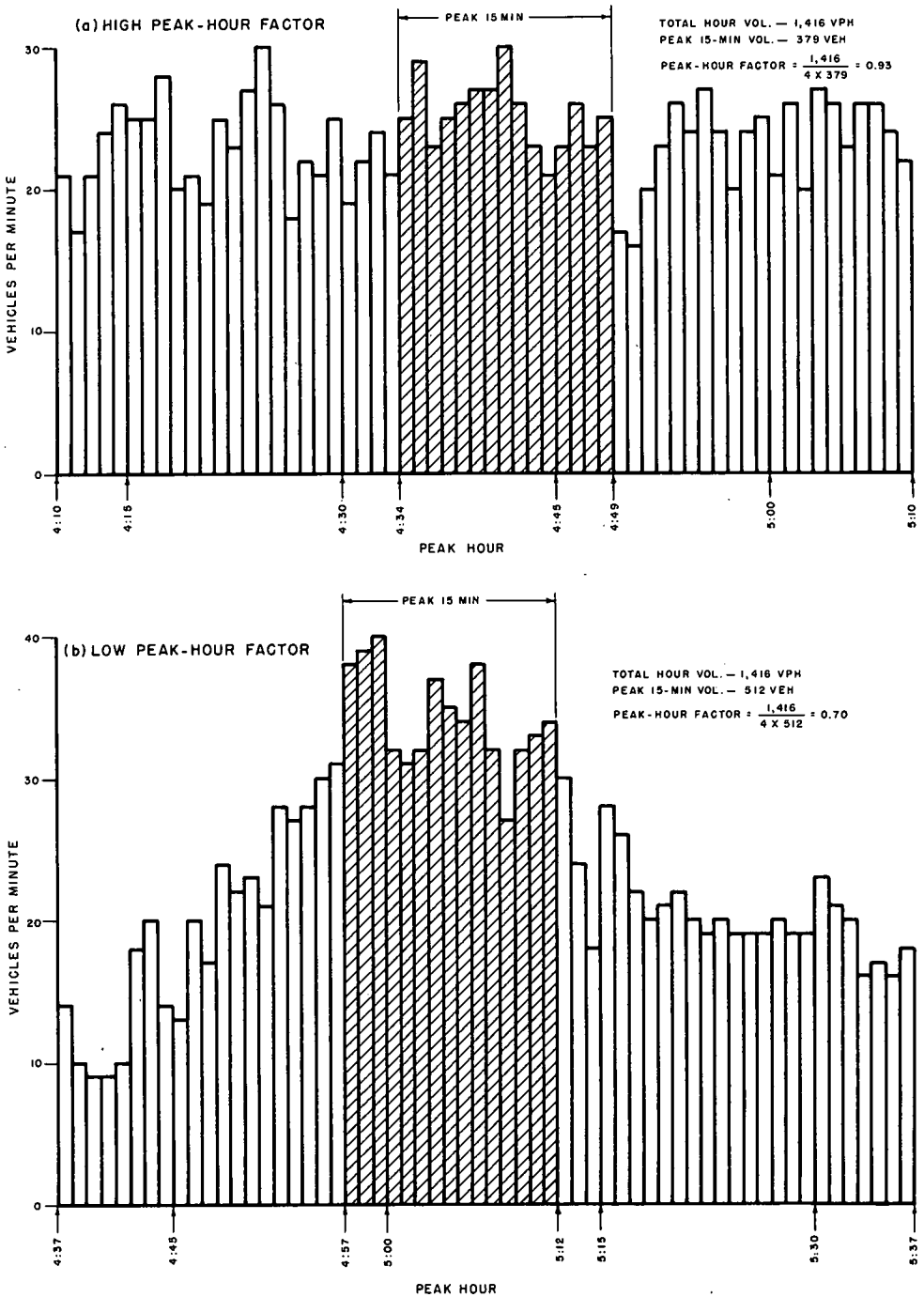
Figure 6.2. *Traffic flow at intersection approach with* (a) *high peak-hour factor and* (b) *low peak-hour factor.*

One-way urban arterial street in downtown area, showing signalized intersection with pavement laning.

counted during the highest 15 consecutive minutes. A separate peak-hour factor must be computed for each approach to the intersection.

The peak-hour factor reflects variations in peaking characteristics of approach roadways and provides a means of more accurately evaluating their operating characteristics.

Figure 6.2 is included here to illustrate the effects of a high or low peak-hour factor. It should be noted that in both cases the peak-hour factor represents the percentage that the area of the 15 shaded bars is to the total area shown. In the case of Figure 6.2a the peak-hour factor is high, indicating that the flow rate was fairly uniform during the peak hour. In Figure 6.2b the peak-hour factor is low, indicating that much of the peak-hour demand arrived during a period much shorter than the full hour.

As defined, it is possible for the peak-hour factor to vary from 0.25 to 1.00. If the traffic flow is entirely uniform during the entire peak hour, so that the peak 15 min carry only one-fourth of the traffic during the hour, the peak-hour factor will be 1.00. At the other extreme, if all the hourly traffic arrives during a single 15-min period, with no traffic during the rest of the hour, the peak-hour factor will be 0.25. It is, of course, highly unlikely that the second condition would ever occur (except, possibly, on a roadway carrying traffic away from a single-purpose parking area).

The lowest peak-hour factor recorded during any of the 1955-6 studies was 0.47. At this location, more than one-half of the hourly flow occurred during the peak 15 min. A peak-hour factor between 0.85 and 0.90 was most common, being observed at 28 percent of the submitted approaches, but about 5 percent were between 0.95 and 1.00, indicating nearly uniform flow throughout the hour.

The peak-hour factor can be determined by the following methods:

1. On-site measurement.—Where operation of an existing intersection is being considered in detail, measurement of peaking characteristics can be accomplished as a part of normal investigation procedures. Relatively detailed data-gathering procedures are required, to provide cycle-by-cycle knowledge of flow rates and characteristics. Manual counting and observation proce-

Separate left-turn lane at high-capacity rural signalized intersection. Note right-turn lane from cross road at extreme left.

dures, photographic techniques, and a variety of electronic recording devices can be utilized.

Ordinary mechanical traffic counters are not adequate for this purpose, because they do not provide the minute-by-minute volumes necessary for computing the peak-hour factor. Utilizing the 15-min counts produced by some counters can result in substantial errors. To illustrate, assume that such a counter recorded the data contained in Figure 6.2b. Analysis of the counter data would probably indicate that the peak hour was from 4:45 to 5:45 and contained slightly less than the total volume shown (assume 1,400 vph). The peak 15 min recorded would be from 5:00 to 5:15, with 467 vehicles (assuming the same accuracy as the minute counts shown). These two values would result in a computed peak-hour factor of 0.75, compared to the actual factor of 0.70. If the true peak was more nearly centered on one of the recording intervals, the error would substantially increase.

2. Similar location measurement.—Peaking characteristics can be measured at a few control locations on traffic corridors in the city, representing the full range of conditions likely to be found in the city. The factor for the most similar location can then be applied to the particular location under study.

3. Estimation.—Often, particularly in the case of general studies of capacity problems over broad areas within a city, it is not feasible to make detailed studies of peaking characteristics. In such cases estimated values may be used, as described in the procedures section of this chapter.

METROPOLITAN AREA POPULATION

One important finding from the analysis of the submitted intersection data was strong indication that approaches in any particular type of area within large metropolitan areas had higher capacities than those of similar geometrics located in equivalent areas in smaller cities. The facts that drivers in large cities are more experienced in coping with high densities and congested traffic conditions, and are more intent on moving through, because they have greater distances

to traverse under these conditions, probably explain this finding.

This effect of metropolitan area size is difficult to determine independently, inasmuch as other primary variables, such as peak-hour factor, are also related to area size. Its effect, nevertheless, has been determined sufficiently for inclusion as one of the variables in the procedures to be described. Nine population groups are included, encompassing a wide range of metropolitan areas from small single cities to widespread urbanized areas composed of several cities.

It should be noted that the intersection data were gathered largely in or near the central cities in the various metropolitan areas studied. Application of the resulting criteria to satellite suburban communities requires judgment to establish whether the community is better considered as a separate independent small city or an outlying portion of the large central city. Local studies may be required to make this determination.

Although few data were gathered for rural areas, the procedures that follow include rural criteria, developed by adaptation of data from the smallest of the population groups to the rural situation.

LOCATION WITHIN METROPOLITAN AREA

For analysis, metropolitan areas are divided into four land use and development classifications or types, as follows:

1. Central business district.—That portion of a municipality in which the dominant land use is intense business activity. This district is characterized by large numbers of pedestrians, commercial vehicle loadings of goods and people, a heavy demand for parking space, and high parking turnover.

2. Fringe area.—That portion of a municipality immediately outside the central business district in which there is a wide range in type of business activity, generally including small commercial, light industrial, warehousing, automobile service activities, and intermediate strip development, as well as some concentrated residential areas. Most of the traffic in this area involves trips that do not have an origin or a destination within the area. This area is characterized by moderate pedestrian traffic and a lower parking

turnover than is found in the central business district, but it may include large parking areas serving that district.

3. Outlying business district.—That portion of a municipality or an area within the influence of a municipality, normally separated geographically by some distance from the central business district and its fringe area, in which the principal land use is for business activity. This district has its own local traffic circulation superimposed on through movements to and from the central business district, a relatively high parking demand and turnover, and moderate pedestrian traffic. Compact off-street shopping developments entirely on one side of the street are not included in the scope of this definition.

4. Residential area.—That portion of a municipality, or an area within the influence of a municipality, in which the dominant land use is residential development, but where small business areas may be included. This area is characterized by few pedestrians and a low parking turnover.

Although the data gathered in the 1955-6 studies provided little information regarding modern off-street shopping centers concentrated on one side of a highway or in one quadrant of an intersection in outlying areas, it can be assumed that the highways serving them fall in this residential category if access to the center is only by marked driveways and businesses do not front on the highways themselves.

The gathered data indicate that:

For *one-way streets without parking* approach capacities are about 10 percent higher in the fringe and outlying business districts than in the central business district. No data are available for such streets in residential areas, because this type of operation rarely occurs, but it is estimated that a 20 percent higher figure would apply.

For *one-way streets with parking on one side* approach capacities are similar in the central and fringe districts. In residential areas this type of street handled approximately 20 percent more traffic than in central and fringe districts. No data are available for one-way streets with parking on one side in outlying business districts, but

it is estimated that the 20 percent value would apply here also.

For *one-way streets with parking on both sides* approach capacities are about 15 percent higher in outlying business districts than in the central business district and the fringe area. In residential areas they are about 25 percent higher than in the central business district.

For *two-way streets, both with and without parking,* approach capacities are about 25 percent higher in all other areas than in the central business district.

There are several reasons for lower capacities in the central business district than elsewhere. Some of the more important ones are (a) a greater frequency of vehicles stopping to load or unload passengers, including both buses and passenger cars; (b) pedestrians causing interference to vehicular traffic; (c) a more circulatory-type traffic flow, involving more turns; (d) presence of appreciable numbers of delivery trucks making brief stops; and (e) curb parking turnover, where parking is present.

Traffic Characteristics

Intersection approach capacity, like the capacity of other highway elements, is influenced by the inherent characteristics of the traffic being accommodated. These characteristics include the amount of turning traffic, the percentage of commercial vehicles, and the operations of local transit buses.

TURNING MOVEMENTS

Turning movements are most directly a traffic characteristic, although they are related to the environment and they also can be, and frequently are, controlled. Certain turning movements at individual intersections can be totally eliminated or otherwise manipulated by traffic engineering techniques for a net increase in capacity at that point. However, as discussed in Chapter Ten, a control which appears desirable in terms of a single location may not be effective in terms of the street system as a whole.

Because of the many interrelationships between turning movements and other traffic and pedestrian movements within the intersection area, most of which have not received detailed study, it is not yet possible to provide refined criteria regarding the effects of turning movements. Listed in the following are some of the more obvious characteristics. In the computation procedures later in this chapter the adjustments presented are partly rationalizations based on such trends and limits of these characteristics as have been identified.

Left turns exhibit the following characteristics:

1. The effect per vehicle on approach capacity is less if two successive vehicles turn left than if single vehicles turn at more widely spaced intervals. It follows that the larger the number of turning vehicles the less the effect per vehicle.

2. The effect of left-turning vehicles is related to the number of opposing vehicles, on two-way streets.

3. The effect of a left turn is dependent on conflicting pedestrian flows, usually those in the crosswalk of the leg into which the turn is being made.

4. A vehicle waiting to make a left turn causes a greater relative reduction in capacity on a narrow street than on a wider street or one having a center dividing island with a left-turn lane.

5. The width of the cross street affects speed and number of vehicles turning (i.e., a wide cross street provides more space to receive left turns and provides a larger turning radius, thus increasing maneuver speed).

Problems of left-turning vehicles should be considered with respect to specific conditions occurring at the particular intersection under study. Treatments such as separate left-turn signal phases may be effective if the number of left-turn movements is high. Leading or lagging green intervals, to allow some turns to clear free of opposing traffic, may be desirable. Channelized left-turn lanes separate left-turning traffic and allow through lanes to move to better advantage, regardless of whether or not the location is signalized. Complete prohibition of left turns at critical points may be desirable at some intersections, if suitable alternate means of handling the movement exist.

Procedures offered in this manual con-

Separate left-turn signal indications and exclusive left-turn lanes at intersection on major urban divided highway.

sider the effect of left turns on capacity in cases where (a) no separate lane or phase is provided for left-turning traffic, (b) a separate lane is provided with no separate signal phase, (c) a separate signal phase is provided with no separate lane, and (d) both a separate lane and a separate signal phase are provided. Although these procedures provide means of assessing the effect of left turns, they still represent only an approximation.

Right turns also influence intersection capacity in varying degrees, depending on conditions at the intersection. Although opposing traffic is not a factor, other influences are much the same as for left turns, including the following:

1. Two or more consecutive vehicles turning have less effect per vehicle than if they arrive separately.

2. Right-turning vehicles are affected by pedestrian movements, usually those in the crosswalk of the leg into which the turn is being made. Sometimes the effect is greater than on left turns, because the conflict is often with large groups of pedestrians at-tempting to leave the curb simultaneously.

3. A vehicle turning right causes a greater relative reduction in capacity on a narrow street than on a wider street.

4. The influence of width of cross street is quite variable. The restrictive effect of a narrow cross street may be greater on right turns than on left, due to the shorter available turning radius. On the other hand, where little pedestrian interference is experienced, where adequate curb return radii are provided, or where continuous right turning is allowed, some studies have shown an increase in capacity with an increase in right turns, particularly where the cross street is wide and turning vehicles clear the intersection more quickly than do through vehicles.

T intersections are a special case. Here, both possible movements involve turns, and the heavier of the two is generally considered as a through movement unless very sharp turns or heavy pedestrian conflicts are encountered.

TRUCKS AND THROUGH BUSES

The presence of trucks tends to reduce intersection approach capacities because their acceleration rates are lower and they occupy more road space, both in length and width, than passenger cars. The degree of this effect varies widely, depending on the type of vehicle, its weight-power ratio, and, in particular, its size and turning characteristics. However, little detailed research has been reported in these various areas separately. Hence, in the computation procedures presented later in this chapter approximate all-inclusive adjustment factors are provided.

For capacity purposes pick-ups, panel trucks, and other light trucks having only four tires are considered as passenger cars, inasmuch as their performance is so similar. All other trucks, from six-tire single units through the largest combinations, as well as through intercity buses and express transit buses, are considered in one category as "trucks and through buses." Passenger car equivalency factors are not used in intersection capacity computations; rather, direct adjustment factors are provided. However, one truck can be considered as equivalent to a minimum of two passenger cars at intersections, even under the best conditions.

Where conditions quite far from average exist, such as where a preponderance of large heavy trucks is present or where substantial numbers of trucks make turns into narrow cross streets, judgment based on local observations should be used in making allowance for the special conditions.

LOCAL TRANSIT BUSES

Local transit buses on urban streets have entirely different effects than through buses, which are considered as trucks. The specific quantitative influence of these effects is presented in this chapter, whereas Chapter Eleven describes transit bus operations more generally.

The specific effect of such buses stopping to load and unload passengers on the capacity of any particular intersection depends on the area of the city, street width, parking conditions, number of buses, and bus stop location. These locations can be grouped into three general categories, as follows:

1. Near-side curb stops—located at the curb on the intersection approach in advance of the intersection proper. Generally, where bus volumes are appreciable a near-side curb bus stop will have a greater adverse effect on intersection capacity than a far-side stop. Both right-turn and through movements will be affected on approaches where no parking is allowed. However, it may offer the advantage to the transit operator of combining delays due to red signals with loading and unloading delays, thus tending to speed up the overall transit operation. This potential advantage to bus operation depends on the buses consistently arriving at the intersection at the beginning of the red signal interval. Detailed studies of bus operations relating to their arrival at the intersection with respect to the red or green signal interval will indicate the practicality of locating the bus stop on the near side of the intersection for this reason.

On approaches where parking is permitted the effect varies, being somewhat dependent on the distance from the intersection that parking would have been restricted if no bus stop existed. If parking otherwise would be permitted near the intersection, the presence of a bus stop provides added capacity for moving traffic, particularly right turns, when not in use.

When traffic movement on the cross street is one-way and approaching the intersection leg of the main street from the right-hand side, the bus stop usually should be located on the near side of the intersection; it thus will produce the least interference with turning movements.

Adjustment procedures for this type of stop are presented later in this chapter.

2. Far-side curb stops—located at the curb immediately beyond the intersection proper on the straight-through exit from the approach under consideration. Far-side bus stops will have only a minor adverse effect on the approach capacity of streets where parking is permitted. This effect is due primarily to buses pulling back into the moving traffic lanes, rather than to loss of the space which they occupy during loading.

On streets without parking the effect is

partially dependent on the percentage of turning traffic. In some cases on wider streets, where turning movements are high, the loss of volume due to the turning movements may nullify any adverse effect. For example, on a three-lane approach and exit, if the left and right turns combined exceed one-third of the total approach volume, the far-side bus stop will probably have little effect on the approach capacity, because the two lanes remaining should accommodate the through flow adequately.

Adjustment procedures are also included in this chapter for this type of stop.

3. Mid-block bus stops—located at the curb at some point in the middle of the block. They are used where the entire street widths near the intersection are needed for moving and storing traffic, where buses must maneuver for a left turn after leaving the stop, or where some other special case exists.

The influence on capacity depends on the circumstances of each individual case, varying from slight (where a stop of adequate length is located in a block having parking elsewhere and no unusual maneuvers are necessary) to substantial (where awkward maneuvers are needed). No general adjustment procedures can be specified, but criteria for corner stops can often be adapted to fit the conditions.

In addition to the three common types just described, other specialized types exist, such as mid-street platforms, for which no general adjustment procedures can be supplied. Local study is necessary in such cases.

The degree to which a bus stop affects intersection capacity depends not only on the stop location, but also on the number of buses using the loading zone during the peak hour, the number of boarding and alighting passengers per bus, and the time taken to accomplish this boarding and alighting. It is obvious that a route with frequent bus service has a considerably greater adverse effect on capacity than one serviced by only one or two buses per hour, but less apparent that a near-side bus stop on a street with parking elsewhere may, under certain conditions, increase capacities above basic "with parking" values. Chapter
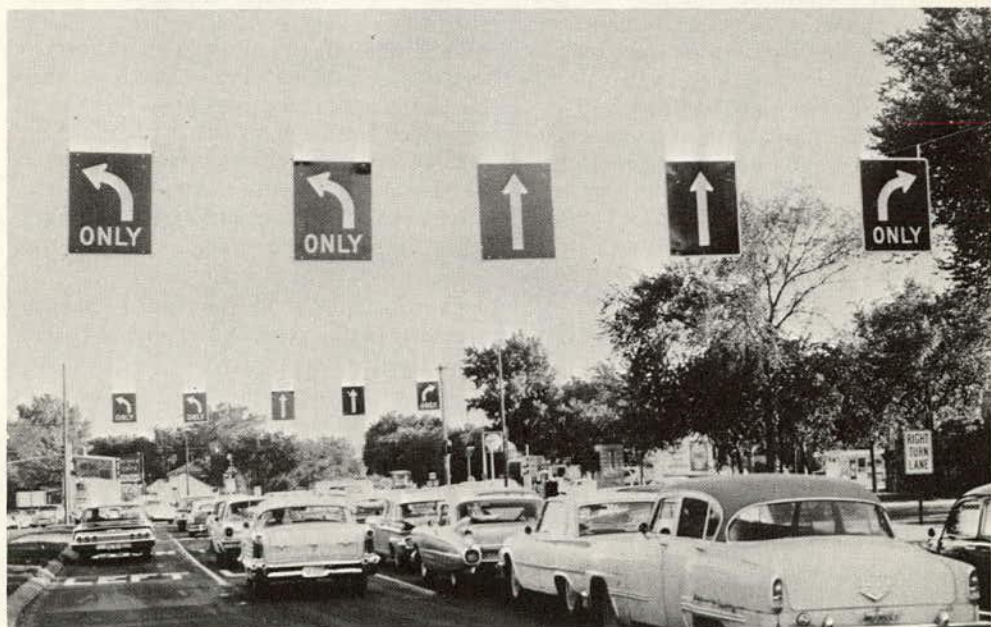
Intersection handling high volume of turns provides for two left-turn lanes, two through traffic lanes, and a right-turn lane.

Eleven discusses this subject of bus stop use characteristics in more detail.

Procedures to be followed in applying average adjustment factors for bus stops are discussed later in this chapter.

Control Measures

Several factors which involve traffic control measures have already been noted in the previous categories. These include parking restrictions, turning controls, and one-way operation. In this section, other traffic control measures which have an influence on intersection capacity are discussed.

TRAFFIC SIGNALS

The ordinary traffic signal regulates traffic flow by displaying a sequence of green (go), yellow (stop if possible), and red (stop) indications to traffic on any given approach. In the simplest case, the same indication is given to all movements on the approach simultaneously, timing is fixed, and there is no interconnection with other signals. In complex installations, on the other hand, each movement may be governed by its own specific series of indications, the timing of each indication may be variable, and the signal very likely is interconnected with others on the street.

It is not the purpose of this manual to discuss signal timing and signal controllers at length; the subject is covered in detail in such references as the "Manual on Uniform Traffic Control Devices" (3) and the "Traffic Engineering Handbook" (4). However, certain fundamentals regarding the influence of signals on capacity and levels of service should be mentioned briefly.

Practically any signal, no matter how timed or controlled, displays periodic red indications when traffic in a certain movement cannot flow. Obviously, these red periods reduce the amount of traffic that can be accommodated in a clock hour (i.e., the capacity per hour), in approximate proportion to their percentage of the total time.

Therefore, "vehicles per hour," referring to an actual clock hour, is not a feasible measure of signalized intersection flow. Instead, "vehicles per hour of green signal indication" is the measure normally used, inasmuch as it largely compensates for the influence of varying percentages of green time assigned to specific approaches.

The main influence of a signal on a particular approach on capacity itself, in terms of vehicles per hour of green, involves the degree to which it stops moving vehicles. At one extreme, if all approaching vehicles in the traffic flow are stopped on the approach before entering (as might occur at an isolated signal not connected with others), then rarely can traffic move away at a rate greater than 1,500 vehicles per hour of green. At the other extreme, if no moving traffic ever is stopped (as would be true at a signal within a perfectly-coordinated progressive signal system), then a capacity flow rate of 2,000 vehicles per hour of green might be achieved. Volumes per actual clock hour would, of course, be proportionately less in either case.

Where level of service is concerned, on the other hand, delays to traffic become important. A single isolated signal with adequate capacity to handle the peak demand on all approaches may nevertheless handle lower volumes on the several legs at widely varying levels of service. Whether or not differing levels on the intersecting streets are desirable often depends on their relative importance. Similarly, a series of signalized intersections, each of which individually has adequate capacity to handle the demand, may nevertheless provide poor service as a group if they are not coordinated, thus forcing traffic to make frequent stops.

Consequently, both individual and coordinated signal control and timing require consideration.

Traffic Signal Control.—A variety of types of traffic signal controllers are utilized, their complexity depending on the purpose they serve. They include (a) pretimed traffic signal control, in which fixed-time signal cycles are established in accordance with predetermined time schedules (usually not exceeding three, to handle morning peak, evening peak, and off-peak); and (b) traffic-actuated signal control, in which the intervals are varied in accordance with the

actuation of demand-detecting devices by vehicles and sometimes also by pedestrians.

The latter are generally divided into two basic types—namely, semi-traffic-actuated control, in which means are provided for actuation by traffic on one or more, but not all, approaches, and fully-traffic-actuated control, in which means are provided for actuation by traffic on all approaches. A variety of detecting devices is used, ranging from simple installations that register only the presence or nonpresence of vehicles on an approach, to complex volume-density equipment in which detector actuations by individual vehicles on each approach, or each lane of each approach, are received, stored, and interpreted.

In the case of an isolated single signal the controller serves simply to allocate the total time available proportionately to the two highways crossing at the particular location, with due consideration to the relative traffic demands and the available pavement widths on the several legs. Increasingly, however, it has been found desirable to interconnect groups of signals, particularly in urban areas, in order to facilitate traffic flow along substantial lengths of an arterial or throughout an entire network of streets. Control of such interconnection ranges from simple linkage of a few signals under one master controller to computer control of the entire signal system of a city.

The capacity computation procedures that follow in this chapter are based on individual signalized locations along highways and streets having only average levels of coordination with other locations along the highway. Application of these procedures to substantial lengths of urban streets involves consideration not only of the individual approach capacities along the route, but also of the overall level of service provided from one end to the other, sometimes including the influence of effective progression (precise coordination designed to keep platoons of traffic continuously moving, never encountering a red signal). These considerations are covered in Chapter Ten.

Signal Timing.—The timing pattern installed on the traffic signal controller at any single intersection has a great influence on the actual number of vehicles that can be accommodated on the various approaches to the intersection in any given time period. Although the basic computational measure used later in this chapter is simply the proportion of the hour when the signal is green for the approach under study, other aspects of timing affect capacity and should be considered. Efficient use of the time available, including application of overlapping phases to the maximum extent possible, can significantly increase the time available for individual movements, thus increasing the actual per-hour capacity of these movements and of the intersection as a whole, even though the per-hour-of-green capacities of the several movements may not change.

Significant elements involved in signal timing include:

1. Cycle length—the total time taken for display of all of the several indications provided by a signal. Cycle lengths are based on total intersection requirements. Generally they should be kept as short as feasible commensurate with accomplishment of all of the individually-phased movements necessary in the total intersection operation. Typical off-peak cycle lengths usually range from 50 to 60 sec. It is seldom feasible to operate on cycle lengths of less than 40 sec or on phase lengths for individual movements of less than 15 sec. Cycle lengths longer than 60 sec may be required at times to accommodate multiple-phase movements at complicated intersections, to provide longer green signal time on those approaches which carry peak flows, or to operate several intersections simultaneously. However, long cycles tend to increase total intersection delay (mainly by producing excessive back-up of traffic on the minor cross street), create fewer opportunities for left-turn movements against opposing traffic at the ends of phases, and increase the problems of pedestrian control (because of the long waiting periods required between pedestrian movements).

Fundamentally, then, maximum efficiency is attained with the shortest feasible cycle length. In practice, however, this shortest feasible length may be quite long in some cases. Careful analysis is necessary in selecting peak-hour cycle lengths and splits to apportion the available time among the

various approaches in such a way as to achieve balanced effective utilization of the green intervals on all approaches.

2. Green time to cycle time ratio (G/C ratio)—a highly important factor used in a capacity calculation to convert vehicles per hour of green to actual vehicles per hour. This ratio is easily computed for pretimed control. Except in the case of traffic-actuated signals, the cycle length and/or split normally is not changed within the peak hour so that the green interval for any phase, divided by the total cycle length, provides the ratio for the approaches which move during that interval.

In the case of traffic-actuated signals, the G/C ratio is not fixed. To obtain the G/C ratio for actuated signals, it is desirable to conduct field observations at the intersection under study or, where this is not feasible, at an intersection with similar physical, traffic, and control conditions. Such studies should include measurements of volumes, cycle length, and green phases. However, where such field studies are not feasible a reasonable estimate of this ratio may be obtained for locations having well-timed fully-actuated or volume-density controllers by dividing the average demand volume per lane for the phase under consideration by the average demand volume per lane for all phases at the intersection, and prorating the results to total 1.00, less an appropriate allowance for yellow signal time. This is usually taken as 0.05 of the cycle per yellow interval in preliminary computations. Because all phases usually do not peak at the same time and are not always fully utilized, it is incorrect to assume that an actuated signal ever operates on a fixed cycle length even under heavy conditions (i.e., all phases reaching the maximum interval, or performing in the same relative pattern, during each cycle).

Semi-actuated controllers present a special situation. Inasmuch as demand is not registered on all legs, the procedure just described is not realistic. Estimation of the average G/C ratio requires consideration of the individual characteristics, such as minimum and maximum green times, of the particular installation.

3. Yellow interval—indication in the sig-

nal sequence of green-yellow-red to warn the motorist that the red (stop) interval will appear in a very short time. Modern regulations require that the motorist stop rather than enter the intersection on the yellow interval unless he is so close to the intersection that such a maneuver is impossible. Once the motorist has entered the intersection legally on the yellow interval, sufficient clearance time must be provided to allow him to exit from the intersection before cross traffic is allowed to enter it. The safest, most efficient operation is now believed to result when the yellow interval is held to a fixed 2 or 3 sec at all intersections regardless of the time required to clear the intersection. The additional time required for the motorist to exit from the intersection should be obtained by providing an all-red interval, again usually 2 or 3 sec, immediately following the yellow interval. The total time of the yellow and all-red periods should be held to the minimum necessary to clear the intersection.

Yellow intervals have not been included in G of the G/C ratio just described because the charts used in the computational procedure were developed on the basis of green time only, recognizing the fact that a few vehicles actually would pass through during the yellow time. It was accepted that in all field data submitted the volume recorded as passing through during any given phase was the total which moved through, regardless of whether the signal was green or yellow, and regardless of the legality of the movements recorded on yellow. Thus, the charts represent average conditions.

MARKING OF APPROACH LANES

As mentioned previously, overall width of approach rather than number and width of lanes has proved to have the most significant influence on capacity. Nevertheless, certain relationships between number of marked lanes and capacity have been detected.

Figure 6.3 shows grouped data relating to the relative efficiencies (expressed in vehicles per hour of green) of different numbers of lanes on various approach widths.

Figure 6.3a, relating to two-way streets

TABLE 6.2—OPTIMUM NUMBER OF LANES FOR VARIOUS APPROACH WIDTHS ON TWO-WAY STREETS WITH NO PARKING

APPROACH WIDTH (FT)	NO. OF TRAFFIC LANES
Up to 17	1
18 to 25	2
26 to 39	3
40 to 55	4

without parking, indicates that various widths accommodate optimum traffic volumes with the number of lanes specified in Table 6.2.

Similar conclusions may be derived from Figure 6.3b for two-way streets with parking, and from Figure 6.3c for one-way streets.

Figure 6.3 and Table 6.2 are guides based on the 1955-6 study data. They are not used directly in capacity determinations, but may serve to indicate how best to operate a given pavement width to assure attainment of computed capacity. Other practical considerations, such as percentage of trucks in the traffic stream, also affect the determination. It would generally be considered undesirable to have less than a 10-ft lane where truck or bus volumes are significant.

INTERSECTION CAPACITY, SERVICE VOLUMES, AND LEVELS OF SERVICE

As is true of all other highway elements, any at-grade intersection approach has a capacity which represents the maximum number of vehicles that can be accommodated given the particular geometrics, environment, and traffic characteristics and controls. Also, as is true for all other elements, operation is far from satisfactory to most drivers at capacity, with substantial approach delays likely.

Approximate levels of service can be described for individual intersections. However, because by definition true level of service is an indicator of the type of operation over a distance, such "point" levels of service obtained at separate intersections are
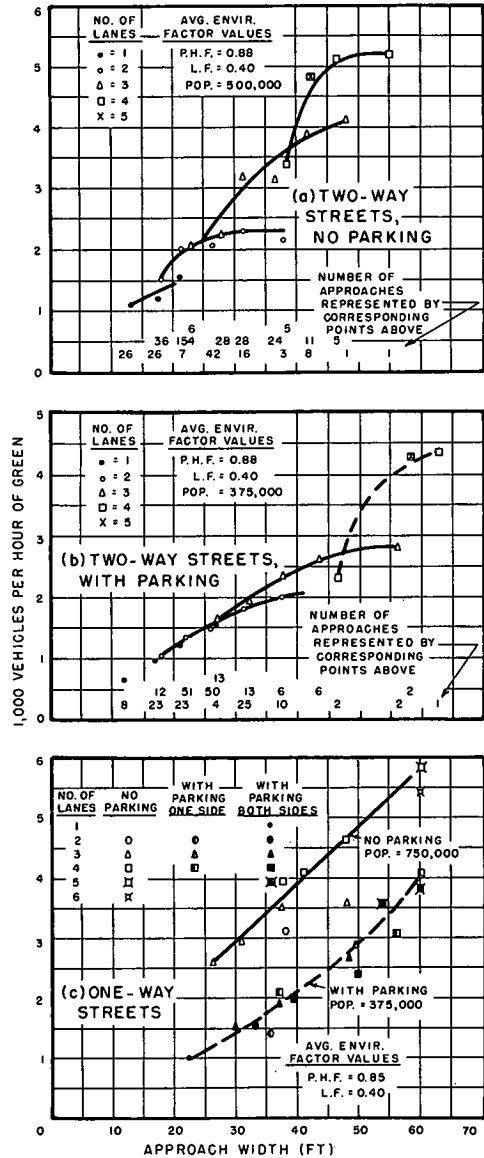
Figure 6.3. Effect of number of lanes on (a) two-way streets with no parking, (b) two-way streets with parking, and (c) one-way streets.

of only local interest. As described in Chapter Ten, substantial lengths of signalized streets must be examined to establish meaningful levels of service.

Although speeds are used as a measure of level of service for most other elements, they are of little use in measuring this element, which produces intentional stops; therefore, some other measure is required. Inasmuch as level of service is described in terms of driver satisfaction, the substitute measure should be some factor that the driver himself sees and interprets in terms of degree of congestion. Of the several factors that have been discussed in the previous section, probably load factor is most evident to the average driver. Hence, it is the best measure of level of service at individual intersections with no or only average signal coordination. (The peak-hour factor, although highly important to the administrator and traffic engineer and rather directly related to the load factor, is a less desirable measure because individual drivers do not see clear-cut evidence of it. Its relationships are shown in Chapter Ten.) For the purpose of defining typical intersectional levels of service, then, load factor is employed.

The conditions which the driver is likely to encounter at each level are next described. This discussion applies to typical signalized intersections, not to perfect or near-perfect progressive signalization, a special case which must be considered over a length of highway as described in Chapter Ten. Throughout the entire range of levels it should be realized that some vehicles will arrive during a red indication and will have to stop. If a reasonably good progression has been established, such vehicles may be few; otherwise, there will be a considerable number simply due to the random pattern of arrivals. For any single intersection, then, even the highest level of service may involve some stops.

At level of service A there are no loaded cycles (i.e., the load factor is 0.0) and few are even close to loaded. No approach phase is fully utilized by traffic and no vehicle waits longer than one red indication. Typically the approach appears quite open, turning movements are easily made, and nearly all drivers find freedom of operation, their only concern being the chance that the light will be red, or turn red, when they approach.

Level of service B represents stable operation, with a load factor of not over 0.1; an occasional approach phase is fully utilized and a substantial number are approaching full use. Many drivers begin to feel somewhat restricted within platoons of vehicles. Under typical rural conditions this frequently will be suitable operation for rural design purposes.

In level of service C stable operation continues. Loading is still intermittent, but more frequent, with the load factor ranging from 0.1 to 0.3. Occasionally drivers may have to wait through more than one red signal indication, and back-ups may develop behind turning vehicles. Most drivers feel somewhat restricted, but not objectionably so. In the absence of local conditions dictating otherwise, this is the level typically associated with urban design practice.

Level of service D encompasses a zone of increasing restriction approaching instability in the limit when the load factor reaches 0.70. Delays to approaching vehicles may be substantial during short peaks within the peak period, but enough cycles with lower demand occur to permit periodic clearance of developing queues, thus preventing excessive back-ups.

Capacity occurs at level of service E. It represents the most vehicles that any particular intersection approach can accommodate. Although theoretically a load factor of 1.0 would represent capacity, in practice full utilization of every cycle is seldom attained, no matter how great the demand, unless the street is highly friction-free. A load factor range of 0.7 to 1.0 is more realistic. In the absence of a local determination, use of 0.85 is recommended for isolated intersections. For interconnected signals a higher factor may be appropriate, as discussed in Chapter Ten. At capacity there may be long queues of vehicles waiting upstream of the intersection and delays may be great (up to several signal cycles).

Level of service F represents jammed conditions. Back-ups from locations downstream or on the cross street may restrict or prevent movement of vehicles out of the approach under consideration; hence, volumes carried are not predictable. No load factor can be established, because full utili-

TABLE 6.3—LEVELS OF SERVICE AND MAXIMUM SERVICE VOLUMES FOR INDIVIDUAL ISOLATED INTERSECTION APPROACHES

LEVEL OF SERVICE	TRAFFIC FLOW DESCRIPTION	LOAD FACTOR
A	Free flow	0.0
B	Stable flow	$\leqslant 0.1$
C	Stable flow	$\leqslant 0.3$
D	Approaching unstable flow	$\leqslant 0.7$
E [a]	Unstable flow	$\leqslant 1.00$
F	Forced flow	— [b]

[a] Capacity.
[b] Not applicable.

zation of the approach is prevented by outside conditions.

Table 6.3 summarizes these intersection level of service criteria.

On the average, the intersection approaches reported during the 1955-6 studies showed a load factor of about 0.40 during the full peak hour, which value is near the start of level D. When it is realized that most of the data came from locations selected as the most heavily used, non-jammed, approaches in the particular cities involved, it becomes apparent that 100 percent loading is far less frequently found than is often supposed. That is, although an intersection approach may look heavily utilized to the casual observer, or to a particular driver who regularly passes through during a short peak period, detailed study usually reveals considerable unused capacity over any full peak hour.

As is true with all other highway elements, the importance of short peaks within the hour on intersection approaches can be established only by relating their consequences to local civic and economic problems as a whole. An intersection approach may be fully adequate if drivers arriving during a once-a-day 15-min peak period (such as when a large individual plant closes) willingly accept a temporary back-up as inevitable or at least as more acceptable than expenditure of local funds to eliminate the very short overload period. On the other hand, if this is found unacceptable locally, level of service during the short period will require consideration as related to possible design modifications.

In the computation procedures that follow, level of service is represented principally by a family of curves applying to the type of intersection under study. Although the influence of the many factors just described undoubtedly varies to some extent, depending on the level of service, sufficiently refined data are not yet available to identify the degree of variation. Hence, the adjustment factors presented apply alike to all levels of service.

PROCEDURES FOR ESTIMATING INTER-SECTION CAPACITY, SERVICE VOLUMES, AND LEVELS OF SERVICE

Thus far this chapter has discussed in general terms the broad field of intersection approach capacity and the many elements and factors which influence it. In this section specific procedures are presented for use in establishing actual capacity and service volume values for a wide range of intersection conditions, and in estimating level of service given a known demand.

First, it is essential to repeat that "vehicles per hour of green" is the basic unit used to express the capacity and service volumes of a signalized approach. Given the volume per hour of green time, by applying to this volume the fraction of the total cycle time that the signal is green for a particular movement, one can calculate the number of vehicles that can clear the intersection from that approach during one hour elapsed time.

Also, it should be made clear that intersection capacity, like uninterrupted-flow capacity, is not a precise determination. Wide ranges of observed volumes under heavy flow conditions were reported for apparently similar physical conditions during the 1955-6 studies. Figure 6.4 shows, for each of the categories given in the "Physical and Operating Conditions" section of this
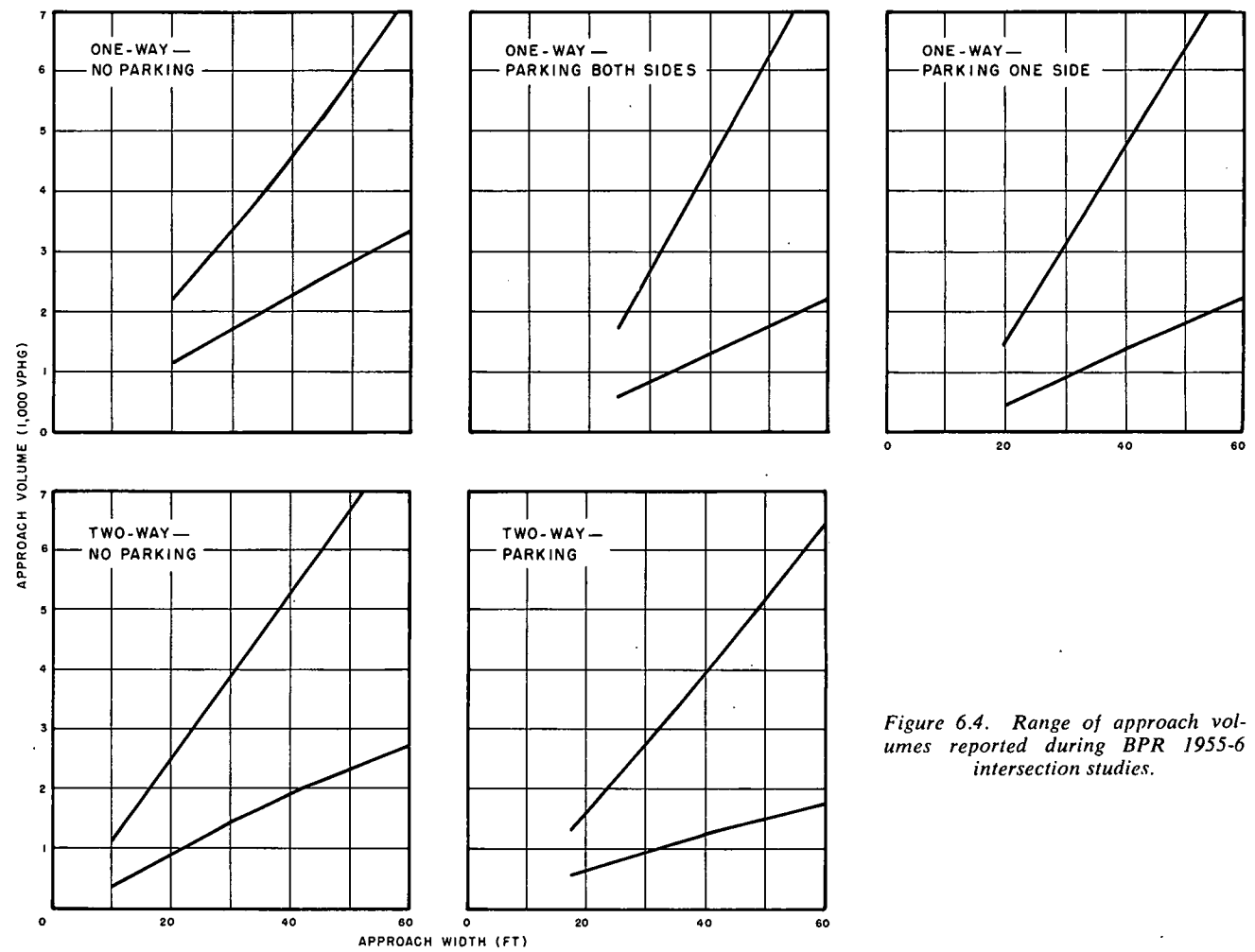
Figure 6.4. Range of approach volumes reported during BPR 1955-6 intersection studies.

chapter, the approximate range in vehicles per hour of green per approach reported in the data submitted. These charts are included only to illustrate the great amount of variance in the data received; they should not be used for computations. Much of this variation has been explained by means of the various factors that have been discussed, but some remains unexplained.

The procedures that follow provide the best overall estimates of intersection capacity and service volumes which can be developed at this time. The user cannot expect, however, to find precise agreement between observed and computed values in every case. Where detailed accurately recorded local observations of an approach have been made while it was operating under fully-loaded, but not jammed, conditions, and the resulting figure is in disagreement with the estimate from this manual, the observed value should be adopted as the capacity of that particular location.

All of the previously noted factors must be considered and evaluated to estimate service volumes and the capacity of an approach under specified operating conditions. Several are consolidated into a set of charts and tables for application; the remainder are considered individually.

Fundamental Capacity Charts and Adjustment Tables

URBAN CONDITIONS

Figures 6.5 through 6.9 and their included tabulations permit fundamental determination of service volumes and capacity on a per-hour-of-green basis, given the width of approach, the load factor, the peak-hour factor, the metropolitan area population, and the location within the metropolitan area. These criteria have been prepared to represent the several categories included in the "Physical and Operating Conditions" section of this chapter; namely, three parking conditions on one-way streets and two parking conditions on two-way streets.

· It should be clearly understood that *these figures do not provide final answers, even when the adjustments contained in the associated tables are applied.* The volume value

as read directly may be suitable for rough comparative computations, but no final problem solution should be read directly from the figures without application of the tabulated adjustments and, usually, others to be described later in this chapter. In particular, this value, being in terms of vehicles per hour of green, is *never* suitable for direct application to an approach, even in approximate solutions. It must *always* be multiplied by the appropriate signal G/C ratio for the approach under consideration to establish the actual capabilities of the approach per hour as signalized.

The approach width used in these figures is the total width of the approach pavement, including any parking lanes present but excluding any separate left-turn or right-turn lanes (i.e., lanes reserved exclusively for turns, either with or without separate signal phases). Where such lanes are utilized the width of approach used for capacity determinations from the charts should be the total width of approach less the width of the exclusive turning lanes. The approach capacity is then increased to account for the effect of these lanes, as described in later sections.

The volume carried, in vehicles per hour of green, represents an average condition with respect to traffic factors. Specifically, it represents 10 percent left turns, 10 percent right turns, 5 percent trucks and through buses, and no local transit buses.

The curves in Figures 6.5 through 6.9 establish the relationship between approach width and vehicles per hour of green for the full range of possible load factors. They are based on selected values of certain factors described in the "Environmental Conditions" section of this chapter; namely, a peak-hour factor of 0.85 (the average value obtained from all data), a metropolitan area population of 250,000, and a downtown location. It is important to note that no assumption is made regarding an average load factor and that only occasionally will the site under study fit the remaining assumed conditions in all respects. Consequently, a choice of load factor always must be made to convert criteria to the actual conditions at the study site. For other values of peak-hour factor and/or metro-

ADJUSTMENT FOR PEAK-HOUR FACTOR AND METROPOLITAN AREA SIZE

METROPOLITAN AREA POP. (1,000'S)	PEAK-HOUR FACTOR						
	0.70	0.75	0.80	0.85	0.90	0.95	1.00
OVER 1,000	0.98	1.04	1.09	1.14	1.20	1.25	1.30
1,000	0.96	1.01	1.06	1.12	1.17	1.22	1.27
750	0.93	0.98	1.03	1.09	1.14	1.19	1.25
500	0.90	0.95	1.00	1.06	1.11	1.16	1.22
375	0.87	0.92	0.98	1.03	1.08	1.14	1.19
250	0.84	0.89	0.95	1.00	1.05	1.11	1.16
175	0.81	0.87	0.92	0.97	1.03	1.08	1.13
100	0.78	0.84	0.89	0.94	1.00	1.05	1.10
75	0.76	0.81	0.86	0.92	0.97	1.02	1.08

ADJUSTMENT FOR LOCATION WITHIN METROPOLITAN AREA

CENT. BUSINESS DIST.	1.00
FRINGE AREA	1.10
OUTLYING BUS. DIST.	1.10
RESIDENTIAL AREA	1.20*

*(EST.)

CONDITIONS:
RIGHT TURNS — 10%
LEFT TURNS — 10%
TRUCKS AND
THROUGH BUSES — 5%
LOCAL TRANSIT
BUSES — NONE

Figure 6.5. Urban intersection approach service volume, in vehicles per hour of green signal time, for one-way streets with no parking.

ADJUSTMENT FOR PEAK-HOUR FACTOR AND METROPOLITAN AREA SIZE

METROPOLITAN AREA POP. (1,000'S)	PEAK-HOUR FACTOR						
	0.70	0.75	0.80	0.85	0.90	0.95	1.00
OVER 1,000	0.99	1.04	1.09	1.14	1.19	1.24	1.29
1,000	0.96	1.01	1.06	1.11	1.17	1.22	1.27
750	0.93	0.99	1.04	1.09	1.14	1.19	1.24
500	0.91	0.96	1.01	1.06	1.11	1.16	1.21
375	0.88	0.93	0.98	1.03	1.08	1.13	1.18
250	0.85	0.90	0.95	1.00	1.05	1.10	1.15
175	0.82	0.87	0.92	0.97	1.02	1.07	1.12
100	0.79	0.84	0.89	0.94	0.99	1.04	1.09
75	0.76	0.81	0.86	0.91	0.97	1.02	1.07

ADJUSTMENT FOR LOCATION WITHIN METROPOLITAN AREA

CENT. BUSINESS DIST.	1.00
FRINGE AREA	1.00
OUTLYING BUS. DIST.	1.20*
RESIDENTIAL AREA	1.20

*(EST.)

CONDITIONS:
RIGHT TURNS — 10%
LEFT TURNS — 10%
TRUCKS AND
THROUGH BUSES — 5%
LOCAL TRANSIT
BUSES — NONE

Figure 6.6. Urban intersection approach service volume, in vehicles per hour of green signal time, for one-way streets with parking one side.

Figure 6.7

ADJUSTMENT FOR PEAK-HOUR FACTOR AND METROPOLITAN AREA SIZE

METROPOLITAN AREA POP. (1,000'S)	PEAK-HOUR FACTOR						
	0.70	0.75	0.80	0.85	0.90	0.95	1.00
OVER 1,000	1.00	1.05	1.09	1.14	1.19	1.24	1.29
1,000	0.97	1.02	1.07	1.11	1.16	1.21	1.26
750	0.94	0.99	1.04	1.09	1.14	1.18	1.23
500	0.91	0.96	1.01	1.06	1.11	1.16	1.21
375	0.88	0.93	0.98	1.03	1.08	1.13	1.18
250	0.85	0.90	0.95	1.00	1.05	1.10	1.15
175	0.82	0.87	0.92	0.97	1.02	1.07	1.12
100	0.80	0.85	0.89	0.94	0.99	1.04	1.09
75	0.77	0.82	0.87	0.92	0.96	1.01	1.06

ADJUSTMENT FOR LOCATION WITHIN METROPOLITAN AREA

CENT. BUSINESS DIST.	1.00
FRINGE AREA	1.00
OUTLYING BUS. DIST.	1.15
RESIDENTIAL AREA	1.25

LOAD FACTOR 1.0
0.7
0.3
0.1
0.0

CONDITIONS:
RIGHT TURNS — 10%
LEFT TURNS — 10%
TRUCKS AND
   THROUGH BUSES — 5%
LOCAL TRANSIT
   BUSES — NONE

APPROACH VOLUME (1,000 VEH PER HR OF GREEN)

CURB-TO-CURB APPROACH WIDTH (FT)

Figure 6.7. Urban intersection approach service volume, in vehicles per hour of green signal time, for one-way streets with parking both sides.

Figure 6.8

ADJUSTMENT FOR PEAK-HOUR FACTOR AND METROPOLITAN AREA SIZE

METROPOLITAN AREA POP. (1,000'S)	PEAK-HOUR FACTOR						
	0.70	0.75	0.60	0.85	0.90	0.95	1.00
OVER 1,000	1.04	1.07	1.11	1.14	1.18	1.21	1.25
1,000	1.01	1.05	1.08	1.11	1.15	1.18	1.22
750	0.98	1.02	1.05	1.09	1.12	1.16	1.19
500	0.96	0.99	1.02	1.06	1.09	1.13	1.16
375	0.93	0.96	1.00	1.03	1.06	1.10	1.13
250	0.90	0.93	0.97	1.00	1.03	1.07	1.10
175	0.87	0.90	0.94	0.97	1.01	1.04	1.07
100	0.84	0.87	0.91	0.94	0.98	1.01	1.05
75	0.81	0.85	0.88	0.92	0.95	0.98	1.02

ADJUSTMENT FOR LOCATION WITHIN METROPOLITAN AREA

CENT. BUSINESS DIST.	1.00
FRINGE AREA	1.25
OUTLYING BUS. DIST.	1.25
RESIDENTIAL AREA	1.25

LOAD FACTOR 1.0
0.7
0.3
0.1
0.0

CONDITIONS:
RIGHT TURNS — 10%
LEFT TURNS — 10%
TRUCKS AND
   THROUGH BUSES — 5%
LOCAL TRANSIT
   BUSES — NONE

APPROACH VOLUME (1,000 VEH PER HR OF GREEN)

CURB-TO-DIVISION-LINE APPROACH WIDTH (FT)

Figure 6.8. Urban intersection approach service volume, in vehicles per hour of green signal time, for two-way streets with no parking.

ADJUSTMENT FOR PEAK-HOUR FACTOR AND METROPOLITAN AREA SIZE

METROPOLITAN AREA POP. (1,000'S)	PEAK-HOUR FACTOR						
	0.70	0.75	0.80	0.85	0.90	0.95	1.00
OVER 1,000	1.00	1.05	1.10	1.14	1.19	1.24	1.29
1,000	0.97	1.02	1.07	1.11	1.16	1.21	1.27
750	0.94	0.99	1.04	1.09	1.13	1.18	1.23
500	0.91	0.96	1.01	1.06	1.11	1.15	1.20
375	0.89	0.93	0.98	1.03	1.08	1.12	1.17
250	0.86	0.91	0.95	1.00	1.05	1.10	1.14
175	0.83	0.88	0.92	0.97	1.02	1.07	1.11
100	0.80	0.85	0.90	0.94	0.99	1.04	1.09
75	0.77	0.82	0.87	0.91	0.96	1.01	1.06

ADJUSTMENT FOR LOCATION WITHIN METROPOLITAN AREA

CENT. BUSINESS DIST.	1.00
FRINGE AREA	1.25
OUTLYING BUS. DIST.	1.25
RESIDENTIAL AREA	1.25

LOAD FACTOR 1.0
0.7
0.3
0.1
0.0

APPROACH VOLUME (1,000 VEH PER HR OF GREEN)

CURB-TO-DIVISION-LINE APPROACH WIDTH (FT)

CONDITIONS:
RIGHT TURNS—10%
LEFT TURNS—10%
TRUCKS AND
THROUGH BUSES—5%
LOCAL TRANSIT
BUSES—NONE

Figure 6.9. Urban intersection approach service volume, in vehicles per hour of green signal time, for two-way streets with parking.

ADJUSTMENT FOR PEAK-HOUR FACTOR

PEAK-HOUR FACTOR=0.70 (AVERAGE RURAL CONDITIONS)	PEAK-HOUR FACTOR=1.00 (RECREATION ROUTE DURING PEAK FLOW)
ADJUSTMENT = 1.00	ADJUSTMENT = 1.40

LOAD FACTOR 1.0
0.7
0.3
0.1
0.0

APPROACH VOLUME (1,000 VEH PER HR OF GREEN)

CURB-TO-DIVISION-LINE APPROACH WIDTH (FT)

CONDITIONS:
RIGHT TURNS — 10%
LEFT TURNS — 10%
TRUCKS AND
THROUGH BUSES — 5%

Figure 6.10. Rural intersection approach service volume, in vehicles per hour of green signal time, for two-way highways with no parking on the traveled way.

politan area population, and for location classifications other than downtown, the table associated with each chart supplies the adjustment factors by which the volume obtained from the chart should be multiplied.

Choice of load factor depends primarily on the intersection level of service desired. For service volumes below capacity the limiting values given in Table 6.3 can be used where more specific local factors are not available.

As mentioned earlier, it is not considered feasible to use a load factor of 1.00 except in those relatively unusual cases where true loading is proven to be continuous throughout the hour. In most actual cases a capacity based on a factor of 1.00 would prove unattainable in practice. Local studies are recommended to establish appropriate values for specific communities. Such studies might even indicate a factor of less than 0.7, in which case the factor for level D should be reduced somewhat. In the absence of such studies, however, a load factor at level E of 0.85 is recommended for isolated intersections and those with only average coordination. Where highly efficient progression exists a factor of 0.95 to 1.00 may prove appropriate, as discussed in Chapter Ten.

Similarly, it is suggested that local investigations be conducted to establish the appropriate level of service for design purposes, with due consideration to the needs of the particular community. Again in the absence of such study results, a load factor of 0.3, representing level C, is considered suitable for average conditions.

Choice of peak-hour factor also must be made. Earlier in this chapter methods of developing local knowledge of this factor were described. Where such local knowledge is not available, estimated factors can be used, as follows:

1. Where long lines of waiting vehicles are typically present or expected at important intersections along major streets in the area throughout the peak hour, a peak-hour factor of 0.90 to 0.95 may be used. (A peak-hour factor of 1.00 should be used only in those rare cases where an un-

usually consistent demand exists throughout a full hour).

2. Where an approach is expected to carry high loads for most of an hour, a peak-hour factor of 0.85 is a reasonable estimate. This value also is a reasonable average for use in the absence of any knowledge about conditions at the study site.

3. Where a major generator provides a high rate of flow over a short period a factor of 0.60 to 0.70 should be considered.

It should be realized that a capacity based on a load factor and/or a peak-hour factor of the order of 0.7 is well below the physical capabilities of the approach, just as the actual capacity of a freeway having a low peak-hour factor is below its physical capabilities. In both cases it is the most traffic that can be reasonably accommodated under the prevailing conditions.

The method of using Figures 6.5 through 6.9 is quite straightforward, once all of the environmental conditions have been established. In the basic case of a given approach width, with volume required, that width is located on the lower scale, and a vertical line is projected up to the appropriate load factor curve. From that point, a horizontal line is then projected across to the volume scale, and the value there obtained is adjusted as necessary by means of the appropriate environmental condition factors from the related tables.

RURAL CONDITIONS

Few data were obtained from rural locations during the 1955-6 studies. However, rationalization based on intersection performance in small cities and known characteristics of rural flows has permitted development of a rural intersection approach service volume chart (Fig. 6.10). Conditions assumed include no parking on the traveled way and a peak-hour factor of 0.70.

Where the rural intersection is on a recreational route, or other route free of typical urban frictional elements but subject to occasional heavy demand for a period of several hours (PHF at or near 1.00), producing a long continuous backlog of vehicles, operation may approach the maximum rate of 1,500 passenger cars per lane

per hour of green at which vehicles, once stopped, can again get under way. The volumes read from the chart, probably for a load factor nearing 1.0, should be multiplied by 1.4 under these conditions.

In the unusual case where parking exists, Figure 6.9 can be used instead of Figure 6.10, but without application of any of the tabulated adjustments. (This, in actuality, approximates application of the factors for a peak-hour factor of 0.70, a population of 75,000, and a residential area, which results in an adjustment of very close to 1.00). Again, for recreational route peaks multiplication by 1.4 is appropriate.

Additional Adjustment Factors

Although application of the fundamental figures and tables is complete at this point, the determination of approach service volumes and capacity remains far from complete. Several other adjustment factors must be considered, one being the all-important G/C ratio, and the remainder being traffic characteristics. All are applied as multipliers. The purpose of each is described in the following.

G/C RATIO

As previously discussed under "Control Measures," the G/C ratio is a highly essential adjustment, reflecting the percentage of the total cycle time during which a green signal is displayed on the approach under consideration. Its use is mandatory in every problem involving through traffic at a signalized intersection approach, because only in a special case (such as a turn lane with a continuous green arrow) will 100 percent green time occur.

As previously mentioned, G represents only actual green or "go" time. It does not include yellow time, no matter whether the yellow indication is displayed alone (properly) or together with green, even though it is recognized that a small portion of the flows calculated will typically move during this period.

When a complete intersection is being analyzed, care must be taken to assure that unintended overlaps of time do not occur. For example, once the needed G/C ratio for one approach is determined, the cross-

street's ratio cannot be taken as simply the difference from 100 percent, inasmuch as allowance must be made for yellow time. For simplicity, arbitrary allowance of 10 percent of the total time for yellow time, 5 percent for each street, usually is satisfactory during preliminary computations involving ordinary four-leg intersections. When the cycle is finally established the actual yellow intervals must, of course, be used. It is recommended that a 3-sec yellow interval normally be allowed. In cases where further clearance is required, such as through very wide or "dog-leg" intersections, an all-red period should be provided in addition.

TURNING MOVEMENTS

Turns—Basic Case; No Separate Turning Lanes or Signals.—Adjustments for the percentage of turning movements performed at simple approaches without special turning lanes or signal indications for turns, reflecting the several effects previously discussed, are given in Table 6.4, which is primarily for right turns but also is applicable to left turns from one-way streets, and Table 6.5, which is for left turns from two-way streets.

It should be noted that the adjustment varies, depending on the width of the street and on whether or not parking is present. In this connection the adjustments for the wider streets in Table 6.4 deserve special mention. It now appears that on intermediate-width streets the adverse influence of right turns is greatest at the 20 percent level; above that level, their adverse influence gradually disappears, probably because for all practical purposes the lane becomes an exclusive turning lane and frictional interferences are largely overcome. On very wide streets it now appears that right turns have little or no adverse influence.

Turns with Separate Turning Lanes and/ or Separate Signal Indications.—At many locations on modern highways and streets specific lanes on intersection approaches are designated for turning movements. These may or may not be in addition to the basic width of the roadway, and they may or may not be controlled by separate signal indications. Similarly, on occasion separate signal indications may be provided without a reserved lane.

For these special cases Tables 6.4 and 6.5 are not applicable. Instead, the special procedures given in the following should be applied. It is assumed here that the turning lanes provided are long enough to handle the turning volumes computed. In practice, the turning lane length should be approximately twice that necessary to handle the average turning volume per cycle, to accommodate random peaks.

Separate Turning Lanes (Signal-Controlled).—This is probably the most common case. The following procedure assumes that no through traffic uses the separate lanes, that pedestrians are controlled so as not to interfere significantly, and that in the case of right-turn lanes adequate curb radii are provided for easy turns. Under these conditions the procedure is equally valid for lanes within the basic roadway width and for added lanes. The steps are as follows:

1. Deduct the width of the reserved lane or lanes from the total approach width. Compute the service volume of the remaining width by means of the basic procedure for intersection approaches, with 0 percent turning movements inserted for the movement or movements accommodated by the reserved lanes.

2. Consider each special turning lane as having the following service volumes per 10 ft of width:

Level	Veh per Hr of Green	Assumed Trucks (%)
A, B, C	800	5
D	1,000	5
E (capacity)	1,200	5

Where two or more turning lanes are provided to handle a particular movement, the additional lanes each should be assigned a service volume of 0.8 times the above values. Apply the appropriate G/C factor for the separate signal indication, and adjust for trucks by means of the factor in Table 6.6.

3. Add the service volumes computed in Steps 1 and 2 to obtain the total for the approach.

Note: This computation yields the physical capabilities of the location, given known signal timing data. However, in many typical applications the G/C ratio will not be known in advance. Instead, given the traffic volumes and distribution (through, left, and right) of the vehicles in the approach, the reverse procedure should be carried out to determine the amount of green time required on the turning lanes. In practice this distribution of demand may well limit the number of vehicles with intent to turn that can reach the intersection in any given cycle to some volume less than the computed value.

If through vehicles as well as turning vehicles use the separate lanes, as might be the case where the roadway is widened on both the near and far sides of the intersection, the foregoing special procedures should not be used. In such cases the basic intersection capacity computations should be applied to the entire approach width, incrementally for each differing combination of signal indications. The left turns should be considered as from one-way streets.

Separate Turning Lanes (No Separate Signal Control).—The following steps are involved:

1. Deduct the width of the reserved lane or lanes from the total approach width. Compute the service volume of the remaining width by means of the basic procedures for intersection approaches, with 0 percent turning movements inserted for the movement or movements accommodated by the reserved lanes.

2a. For a right-turn lane (of adequate length): For any level, use $600 \times G/C$ vehicles per hour assuming 5 percent trucks, if the turns must be made simultaneously with pedestrian crossings. If pedestrians are not present, use the values given for the signal-controlled case. In either case, adjust for trucks by means of Table 6.6.

2b. For a left-turn lane (of adequate length): For any level, consider the service volume, in passenger cars, as the difference between 1,200 vehicles and the total opposing traffic volume in terms of passenger

cars per hour of green, but not less than two vehicles per signal cycle.

3. Add the service volumes computed in Steps 1, 2a, and 2b to obtain the approach service volume.

Note: As in the previous case, this computation establishes physical capacities. In actual applications, with given distributions of through, left, and right traffic, the volume

of through traffic may limit the possible supply of turning traffic which can reach the intersection to some value less than the computed capacity.

Separate Signal Control (No Separate Lane).—This situation is found where certain turning movements are permitted for times different from the basic phase

TABLE 6.4—ADJUSTMENT FACTORS FOR RIGHT TURNS ON TWO-WAY STREETS,[a] RIGHT TURNS ON ONE-WAY STREETS,[a] AND LEFT TURNS ON ONE-WAY STREETS [a]

| TURNS [b] (%) | ADJUSTMENT FACTOR | | | | | |
| | WITH NO PARKING [c] | | | WITH PARKING [d] | | |
	APPROACH WIDTH ≤15 FT	APPROACH WIDTH 16 TO 24 FT	APPROACH WIDTH 25 TO 34 FT	APPROACH WIDTH ≤20 FT	APPROACH WIDTH 21 TO 29 FT	APPROACH WIDTH 30 TO 39 FT
0	1.20	1.050	1.025	1.20	1.050	1.025
1	1.18	1.045	1.020	1.18	1.045	1.020
2	1.16	1.040	1.020	1.16	1.040	1.020
3	1.14	1.035	1.015	1.14	1.035	1.015
4	1.12	1.030	1.015	1.12	1.030	1.015
5	1.10	1.025	1.010	1.10	1.025	1.010
6	1.08	1.020	1.010	1.08	1.020	1.010
7	1.06	1.015	1.005	1.06	1.015	1.005
8	1.04	1.010	1.005	1.04	1.010	1.005
9	1.02	1.005	1.000	1.02	1.005	1.000
10	1.00	1.000	1.000	1.00	1.000	1.000
11	0.99	0.995	1.000	0.99	0.995	1.000
12	0.98	0.990	0.995	0.98	0.990	0.995
13	0.97	0.985	0.995	0.97	0.985	0.995
14	0.96	0.980	0.990	0.96	0.980	0.990
15	0.95	0.975	0.990	0.95	0.975	0.990
16	0.94	0.970	0.985	0.94	0.970	0.985
17	0.93	0.965	0.985	0.93	0.965	0.985
18	0.92	0.960	0.980	0.92	0.960	0.980
19	0.91	0.955	0.980	0.91	0.955	0.980
20	0.90	0.950	0.975	0.90	0.950	0.975
22	0.89	0.940	0.980	0.89	0.940	0.980
24	0.88	0.930	0.985	0.88	0.930	0.985
26	0.87	0.920	0.990	0.87	0.920	0.990
28	0.86	0.910	0.995	0.86	0.910	0.995
30 +	0.85	0.900	1.000	0.85	0.900	1.000

[a] No separate turning lanes or separate signal indications.
[b] Handle right turns and left turns separately in all computations; do not sum.
[c] No adjustment necessary for approach width of 35 ft or more; that is, use factor of 1.000.
[d] No adjustment necessary for approach width of 40 ft or more; that is, use factor of 1.000.

TABLE 6.5—ADJUSTMENT FACTORS FOR LEFT TURNS ON TWO-WAY STREETS [a]

TURNS (%)	ADJUSTMENT FACTOR					
	WITH NO PARKING			WITH PARKING		
	APPROACH WIDTH ≤ 15 FT	APPROACH WIDTH 16 TO 34 FT	APPROACH WIDTH ≥ 35 FT	APPROACH WIDTH ≤ 20 FT	APPROACH WIDTH 21 TO 39 FT	APPROACH WIDTH ≥ 40 FT
0	1.30	1.10	1.050	1.30	1.10	1.050
1	1.27	1.09	1.045	1.27	1.09	1.045
2	1.24	1.08	1.040	1.24	1.08	1.040
3	1.21	1.07	1.035	1.21	1.07	1.035
4	1.18	1.06	1.030	1.18	1.06	1.030
5	1.15	1.05	1.025	1.15	1.05	1.025
6	1.12	1.04	1.020	1.12	1.04	1.020
7	1.09	1.03	1.015	1.09	1.03	1.015
8	1.06	1.02	1.010	1.06	1.02	1.010
9	1.03	1.01	1.005	1.03	1.01	1.005
10	1.00	1.00	1.000	1.00	1.00	1.000
11	0.98	0.99	0.995	0.98	0.99	0.995
12	0.96	0.98	0.990	0.96	0.98	0.990
13	0.94	0.97	0.985	0.94	0.97	0.985
14	0.92	0.96	0.980	0.92	0.96	0.980
15	0.90	0.95	0.975	0.90	0.95	0.975
16	0.89	0.94	0.970	0.89	0.94	0.970
17	0.88	0.93	0.965	0.88	0.93	0.965
18	0.87	0.92	0.960	0.87	0.92	0.960
19	0.86	0.91	0.955	0.86	0.91	0.955
20	0.85	0.90	0.950	0.85	0.90	0.950
22	0.84	0.89	0.940	0.84	0.89	0.940
24	0.83	0.88	0.930	0.83	0.88	0.930
26	0.82	0.87	0.920	0.82	0.87	0.920
28	0.81	0.86	0.910	0.81	0.86	0.910
30+	0.80	0.85	0.900	0.80	0.85	0.900

[a] No separate turning lanes or separate signal indications.

length for through traffic, by means of green arrow indications, although reserved lanes for those movements are not provided. It is also found where flows in the two opposing directions on a given street do not have entirely simultaneous green periods. An example is the provision of "leading" or "lagging" green, which in effect provides turns free of opposing traffic for part but not all of the time. Again, separate lanes are not assigned, but the intent generally is to permit an unopposed turning movement. The following steps are involved:

1. Where there is opposing traffic apply the basic intersection capacity computation procedures to the entire approach width, incrementally, for each differing indication combination.

2. Where left turns are unopposed compute each such increment by the basic methods, but consider the left turns as left turns from one-way streets.

3. Add the results of the several incremental steps to obtain the approach service volume.

These alternatives assume that the likelihood of through vehicles becoming "trapped" behind a vehicle intending to turn, within any given increment, is no greater than it would be in an ordinary signal interval of the same characteristics. This assumption is not always valid. For instance, if left turns are permitted only during leading green time and not through opposing traffic during the basic green time, the left lane will be more totally blocked than the general assumption indicates.

TRUCKS AND THROUGH BUSES

Vehicles per hour of green must next be adjusted for the effect of trucks and through buses not making local en route stops. The basic intersection capacity curves represent average urban peak-period conditions, insofar as truck volume is concerned. This was found to be 5 percent trucks. Adjustment for other conditions involves a 1 percent reduction for each percentage point by which trucks exceed 5 percent of the total number of vehicles, or a 1 percent increase for each percentage point that trucks are less than 5 percent of the total vehicles. Table 6.6 gives the correction factors for given truck percentages.

LOCAL TRANSIT BUSES

Local buses—that is, those picking up and discharging passengers at regular posted stops along a street—have a much greater influence on capacity than do through trucks and buses. As mentioned earlier, their effect varies considerably, depending on the type of area, street width, parking conditions, location of the stop (near side of intersection, far side, or other), and number of buses.

Figures 6.11 through 6.14 are nomographs presenting the effects of a range of volumes of local buses, using near-side or far-side bus stops, on streets with and without parking. They provide the necessary adjustment factors for nearly all conditions found in the field. However, certain conditions are not directly covered, primarily mid-block stops, mid-street stops, and stops handling more than 90 buses in downtown areas or 120 elsewhere. In some instances the nomographs can be adapted to handle the mid-block case; but in the remaining situations the special conditions making the unusual installation necessary are such that detailed local study will be required to establish capacities. The nomographs are largely rationalizations developed from limited available knowledge.

The charts are used as follows. Given the number of buses per hour, they are entered at the appropriate point on the vertical scale at the upper left. A horizontal line is projected to the turning line representing the appropriate type of area, and a vertical line is then projected downward to the given number of lanes, if known (or approach width if vehicles do not consistently form the same number of lanes). A

TABLE 6.6—TRUCK AND THROUGH BUS ADJUSTMENT FACTORS

TRUCKS AND THROUGH BUSES (%)	CORRECTION FACTOR	TRUCKS AND THROUGH BUSES (%)	CORRECTION FACTOR	TRUCKS AND THROUGH BUSES (%)	CORRECTION FACTOR
0	1.05	7	0.98	14	0.91
1	1.04	8	0.97	15	0.90
2	1.03	9	0.96	16	0.89
3	1.02	10	0.95	17	0.88
4	1.01	11	0.94	18	0.87
5	1.00	12	0.93	19	0.86
6	0.99	13	0.92	20	0.85
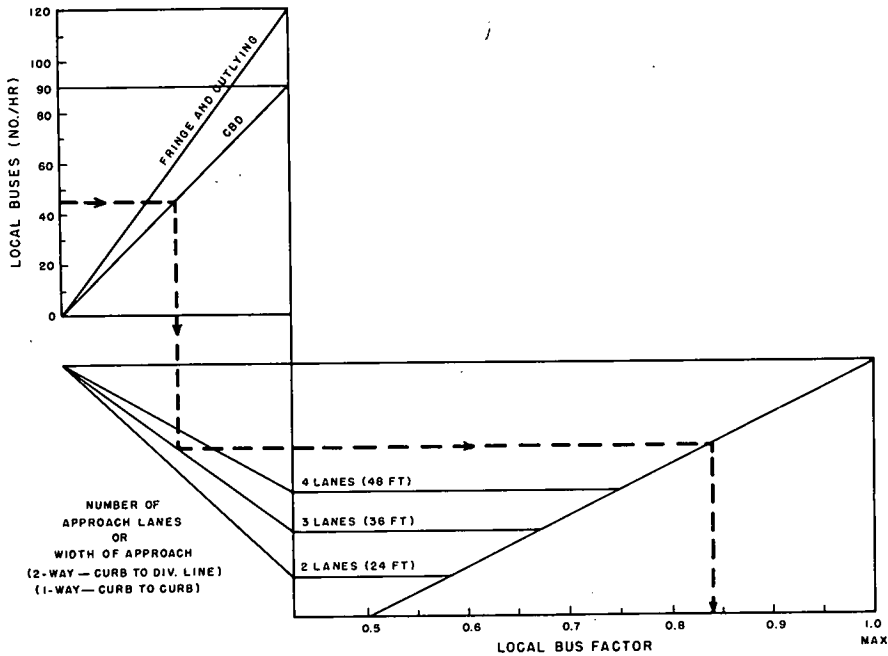
Figure 6.11. Local bus factor for near-side *bus stop on street with* no parking.

further horizontal extension is drawn to a final turning line, representing percentage of turning movements in all but one chart, following which the proper adjustment factor is read from the lower scale. Like the previous factors, it is applied as a multiplier.

In the case of near-side stops on streets with parking present except at the bus stop (Fig. 6.12) the adjustment factor in some instances may be greater .than 1.0. This reflects the secondary .purpose provided by the stop—as a turn lane for moving traffic except when occupied by a bus. In the other three cases no adjustments greater than 1.0 are shown. In these cases, if an intercept with the appropriate turning movements diagonal would be outside the range of the chart (to the right of the right edge) the maximum factor of 1.0 should be adopted.

Also, in the case of near-side stops on streets with parking (Fig. 6.12) a series of three families of turning lines is presented to cover various percentages of turns for two-, three-, and four-lane approaches separately. Where only width of approach is known, the family of turning lines used should be that for the next lower given width increment.

Where widths are greater than those shown on the chart approximate extrapolation is permissable, but extrapolation to greater numbers of buses is questionable.

Interpretations and Applications of
·Procedures

The basic procedures presented thus far describe determination of intersection approach capacities and service volumes, given the width of the approach. They assume that in any specific problem the desired level of service is known, thereby identifying the applicable load factor, which in turn permits selection of the correct curve on the appropriate chart. This is the case in many operational studies where the goal is to determine approach capabilities at pre-established levels of service.
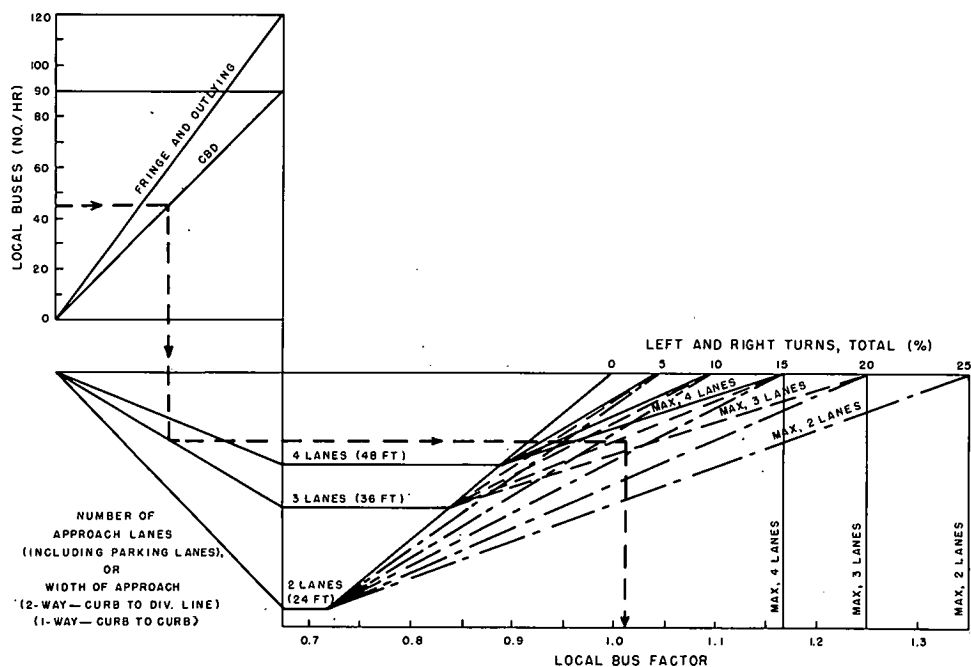
Figure 6.12. Local bus factor for near-side *bus stop on street* with parking.
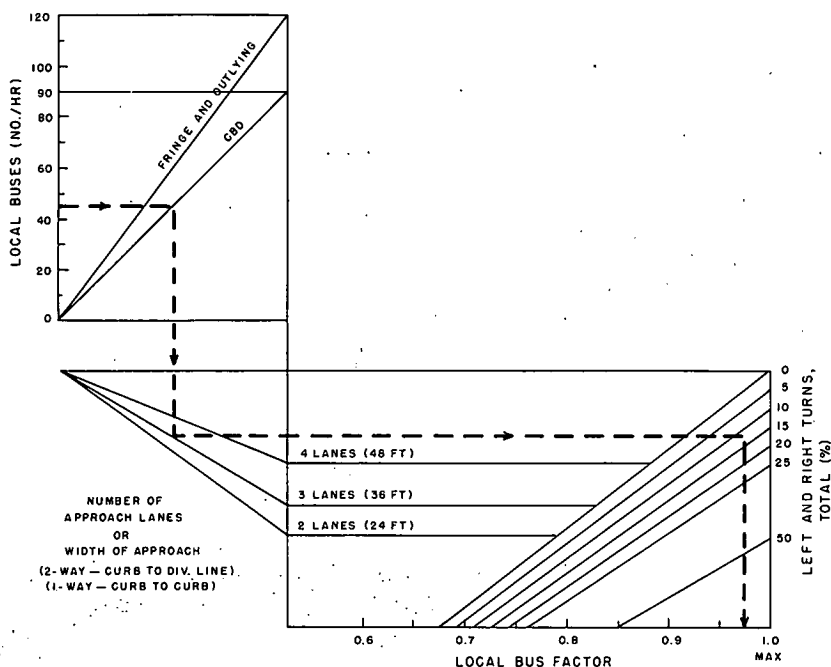
Figure 6.13. Local bus factor for far-side *bus stop on street with* no parking.
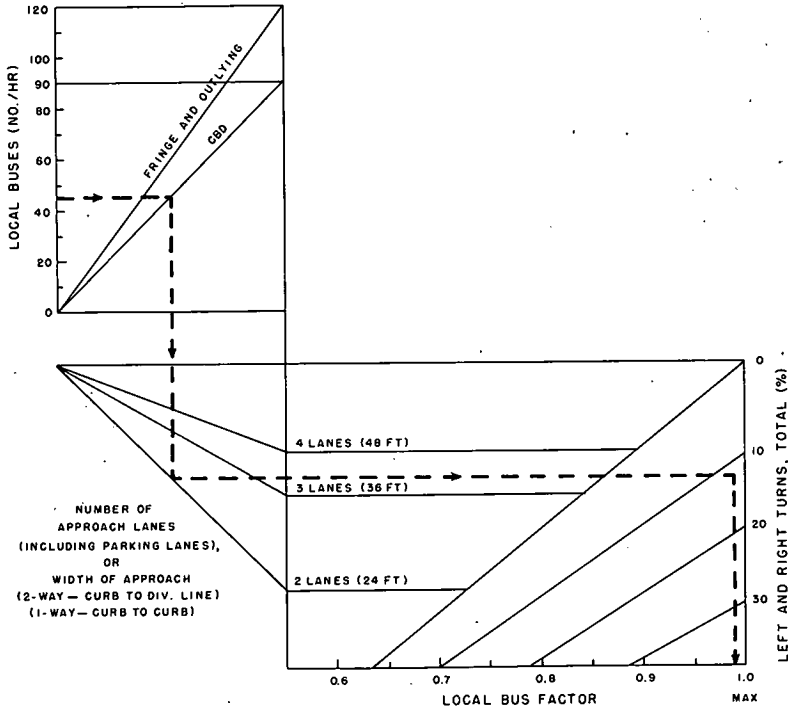
Figure 6.14. Local bus factor for far-side *bus stop on street* with parking.

As long as the desired level of service is known, the procedures can be used "backward" as well as "forward" without difficulty. That is, given a demand volume, that volume can be converted to vehicles per hour of green through division by the several factors previously described. (Care must be exercised here to ensure that *all* such factors, including both those superimposed on the charts and those discussed separately, are considered.) Using the appropriate chart, and the load factor identified by the level of service desired, the required width can be determined. This procedure often is used in development of new designs.

Where, however, level of service is the unknown variable whose value must be determined, it becomes necessary to work toward the plotted curves from both scales. Here, both demand volume and approach width must be known or estimated. Demand volume is then adjusted to vehicles per hour of green. The intersection of lines projected from the volume and width scales identifies the predicted load factor, from which intersection level of service can be specified. This procedure is useful in highway system evaluations where deficiencies are being identified.

In nearly every case the G/C ratio is a special consideration because it in itself is a variable. As the principal link between the particular approach under consideration and the intersection operation as a whole, it is dependent on conditions outside the specific approach under study. In operational problems there will be an existing G/C ratio which can be used as a base. However, in design or new signalization problems a tentative cycle and cycle split must be assumed. Generally, in preliminary com-

putations this approximate split is based
directly on the relative demand volumes on
the several legs and inversely on anticipated
widths available. In the simple case of two
intersecting streets at any given time, one
of the approaches on each will govern that
street, and this becomes:

$$\text{Signal Split Ratio} = \frac{\text{Approach 1 Time}}{\text{Approach 2 Time}} =$$

$$\frac{\text{Vol., Approach 1}}{\text{Vol., Approach 2}} \times \frac{\text{Width, Approach 2}}{\text{Width, Approach 1}}$$

As the solution is gradually refined, width
may be substituted for green time, or vice
versa, or it may be concluded that balanced
levels of service are not desired on all legs.
Each of these considerations will influence
the overall problem. Hence, in practice,
unless the G/C ratio is rigidly established
in advance due to other considerations (such
as the time needed for pedestrians to cross,
where this exceeds time needed for ve-
hicular traffic), a refined solution requires
several trial-and-error steps.

In some cases at existing locations,
knowledge of traffic characteristics (per-
centages of turns and of trucks, and local
bus operations) of the study site may be
meager. If insufficient data are on file from
previous studies to permit even reasonable
approximation of these values, brief field
studies may be required. In the case of new
designs the best available traffic forecasts
must be used.

In the typical problem solutions that fol-
low, the various ways in which these pro-
cedures can be employed are shown.

TYPICAL PROBLEM SOLUTIONS—SIGNALIZED
INTERSECTIONS

EXAMPLE 6.1
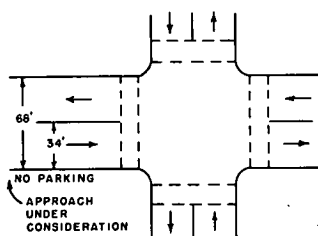
Part a. (To illustrate the use of Figures
6.5 to 6.10 only).

Problem:

Determine the unadjusted number of ve-
hicles per hour of green handled by a two-
way street intersection approach with no
parking, an approach width of 34 ft, and a
load factor of 0.3, in an outlying business
district of a metropolitan area with a popu-
lation of 500,000, and a peak-hour factor of
0.80.

Solution:

Figure 6.8 is the applicable chart
for a two-way street with no parking.
Enter the lower scale of the chart at a width
of 34 ft and project upward to the curve rep-
resenting a load factor of 0.3. Using this
intercept as a turning point, project hori-
zontally to the left-side volume scale. Read
volume of 2,380 vph of green time under
average conditions. From the upper of the
related tables on the figure, a city with a
population of 500,000 and a peak-hour fac-
tor of 0.80 is found to have an adjustment
of 1.02, while from the lower table the ad-
justment for an outlying business district is
taken as 1.25. Multiplying 2,380 by 1.02
and 1.25 results in a service volume of 3,035
vph of green time.

The value obtained from the foregoing is
not a complete solution, but is simply the
result obtained from complete use of the
appropriate basic figure and related tables.
It must then be corrected for signal timing,
turning movements, trucks, and bus stops
(see Part b).

Part b. (To illustrate the completion of
the intersection service volume determina-
tion procedure).

Problem:

Given the intersection approach described
in Part a, determine its actual service vol-
ume under the following conditions:

25 sec of green signal time for all move-
ments during each 60-sec cycle.

15 percent right turns and 5 percent left
turns, with no separate lanes or signal indi-
cations.

2 percent trucks.

45 local buses per hour, utilizing a near-
side bus stop.

Solution:

G/C ratio = 25/60 = 0.42.

From Table 6.4, right-turn adjustment for 15 percent turns from 34-ft approach with no parking = 0.99.

From Table 6.5, left-turn adjustment for 5 percent turns from 34-ft approach with no parking = 1.05.

From Table 6.6, truck adjustment for 2 percent trucks = 1.03.

From Figure 6.11, bus adjustment for 45 buses using near-side stop with no parking in outlying area = 0.87.

Multiply the chart value obtained in Part a by these factors, or 3,035 × 0.42 × 0.99 × 1.05 × 1.03 × 0.87 = 1,190 vph. This is the actual approach service volume under the prevailing conditions, which include a load factor of 0.3, shown in Table 6.3 to be the limit for intersection level of service C. Thus, for these conditions, this is the limiting intersection service volume for level C.

EXAMPLE 6.2
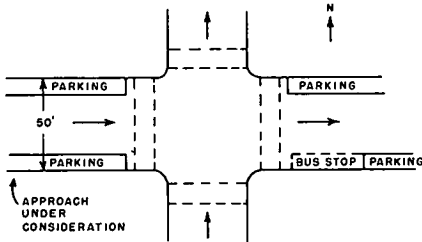
Problem:

GIVEN CONDITIONS:

One-way street, crossing one-way street.

West leg approach under consideration; width = 50 ft.

Parking both sides.

Fringe area.

Metro. area population = 175,000.

Peak-hour factor = 0.75.

Loading = About 10 cycles loaded per hour.

Signal cycle = 60 sec.

Green time = 30 sec per cycle.

Right turns = None; not possible.

Left turns = 8 percent (no separate lane or signal indication).

Trucks = 7 percent.

Local buses = 10 per hour, using far-side stop.

Maximum load factor typically observed in the area, at capacity = 0.90.

DETERMINE:

(a) Service volume being handled under given conditions.

(b) Capacity.

Solution:

(a) Service volume for given conditions. Figure 6.7 applies. For width of 50 ft, and load factor = $\frac{10 \text{ cycles loaded}}{60 \text{ cycles/hr}}$ = 0.17, the chart volume per hour of green = 2,600 vphg. Adjustment for PHF of 0.75 and population of 175,000 (from table on chart) = 0.87. Adjustment for fringe area (from table on chart) = 1.00 (that is, none necessary). Then 2,600 × 0.87 × 1.00 = 2,260 vphg, uncorrected for signal and traffic factors.

G/C ratio = 30/60 = 0.50.

Adjustment for 0 percent right turns on approach over 40 ft wide with parking (from Table 6.4) = 1.00 (that is, none necessary).

Adjustment for 8 percent left turns on approach over 40 ft wide with parking (from Table 6.4) = 1.00 (that is, none necessary). (Note that Table 6.5 was *not* used because a one-way, rather than a two-way, street is under consideration.)

Adjustment for 7 percent trucks (from Table 6.6) = 0.98.

Adjustment for 10 buses per hour, at far-side stop (from Figure 6.14) = 1.00, maximum value (that is, none necessary).

Then, service volume = 2,260 × 0.50 × 1.00 × 1.00 × 0.98 × 1.00 = 2,260 × 0.49 = 1,110 vph. With load factor = 0.17, operation is within intersection level of service C.

(b) Capacity

At capacity, the foregoing computations remain unchanged, with the exception of load factor and, possibly, peak-hour factor.

The given conditions indicate that capacity typically would occur at a load factor of 0.90 in the city involved.

(1) Capacity under present overall demand conditions (PHF = 0.75):

From Figure 6.7, for 50-ft width and load factor = 0.90, the chart volume = 3,800 vphg.

No changes in adjustment factors previously used.

Multiplying by the two combined adjustment factors obtained above, capacity = $3,800 \times 0.87 \times 0.49 = 1,620$ vph~(for LF = 0.9, PHF = 0.75).

In the future, as demand grows in the city, the peak-hour factor will gradually increase; it may in time reach 0.95 (with LF remaining at 0.90).

(2) Capacity under future demand conditions (PHF = 0.95):

Chart volume remains 3,800 vphg.

New adjustment, for PHF of 0.95 and population of 175,000 = 1.07.

Adjustment for fringe area remains 1.00.

Combined adjustment for signal and traffic factors remains 0.49.

Capacity = $3,800 \times 1.07 \times 1.00 \times 0.49 = 1,990$ vph (for LF = 0.9, PHF = 0.95).

Note: In practice, by the time the PHF reaches 0.95, it is likely that other variables would change, including possibly the population itself; hence, the foregoing prediction is probably oversimplified.

EXAMPLE 6.3
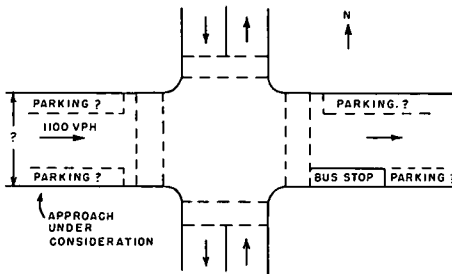
Problem:

GIVEN CONDITIONS:

One-way E-W crossing two-way N-S.
West leg approach under consideration; demand = 1,100 vph.
Central business district.
Metro. area population = 500,000.
Peak-hour factor = 0.90.
Signal cycle = 70 sec.
Green time = 35 sec (which cannot be increased due to cross-street requirements).

Right turns = 3 percent.
Left turns = 5 percent.
Trucks = 4 percent.
Local buses = 20 per hour; far-side stop.
Intersection level of service C desired.

DETERMINE:

Width of approach necessary to handle given demand, at level C, (a) with parking, and (b) without parking.

What is the effective width taken up by parking?

Solution:

(a) With parking.

Convert the demand volume to vehicles per hour of green for the conditions on which the fundamental charts are based. (These will be obtained directly as tabulated, but all must be applied in reciprocal form because the conversion being made is from actual to standard conditions).

For turning movement and local bus factors, an approximate width must be chosen by inspection of the appropriate basic figure (here Fig. 6.7) before the adjustment factors can be taken from Table 6.4. Therefore, it is wise to select the remaining factors first and determine their effect on the volume value, in order to be able to make a reasonably accurate width estimate, which can be easily revised if necessary.

Adjustments not dependent on width:

4 Percent trucks (Table 6.6) = 1.01.

Percent green (G/C ratio) = 35/70 = 0.50.

Metro. area population and PHF (Fig. 6.7) = 1.11.

Central business district (Fig. 6.7) = 1.00.

First trial standardized vphg =
$$\frac{1,100}{1.01 \times 0.50 \times 1.11 \times 1.00} = 1,963 \text{ vphg.}$$

From Table 6.3, for level of service C, load factor = 0.3.

From Fig. 6.7, for load factor = 0.3, the approximate width = 39 ft.

Adjustments dependent on width and parking:

3 Percent right turns (Table 6.4) = 1.015.

5 Percent left turns (Table 6.4) $=1.01$.

20 Local buses (Fig. 6.14, for 39-ft width and 8% total turns) $=1.00$ (max. value).

Second trial standardized vphg $=$

$$\frac{1,963}{1.015 \times 1.01 \times 1.00} = 1,910 \text{ vphg.}$$

From Fig. 6.7, for load factor $=0.3$, width $=38$ ft. Inasmuch as the assumption of width proved to be nearly correct, no re-computation is needed, because the adjustment factors will not change.

Final width, with parking, at capacity $=$ 38 ft.

(b) Without parking.

The procedure is the same as for (a), but uses Figure 6.5 as the base.

Adjustments not dependent on width:
4 Percent trucks (Table 6.6) $=1.01$.
Percent green (G/C ratio) $=35/70=0.50$.
Metro. area population and PHF (Fig. 6.5) $=1.11$.
Central business district (Fig. 6.5) $=1.00$.
First trial standardized vphg $=$

$$\frac{1,100}{1.01 \times 0.50 \times 1.11 \times 1.00} = 1,963 \text{ vphg.}$$

From Fig. 6.5, for load factor $=0.3$, the approximate width $=24$ ft.

Adjustments dependent on width and parking:
3 Percent right turns (Table 6.4) $=1.035$.
5 Percent left turns (Table 6.4) $=1.025$.
20 Local buses (Fig. 6.13) $=0.99$.
Second trial standardized vphg $=$

$$\frac{1,963}{1.035 \times 1.025 \times 0.99} = 1,870 \text{ vphg.}$$

From Figure 6.5, for load factor $=0.3$, the width $=23$ ft. Therefore, the assumption of width is acceptable, as the differences in adjustments would be insignificant.

The final width, with no parking, at capacity $=23$ ft.

Conclusion:

Parking takes up an effective width of $38-23=15$ ft, at level C.

EXAMPLE 6.4
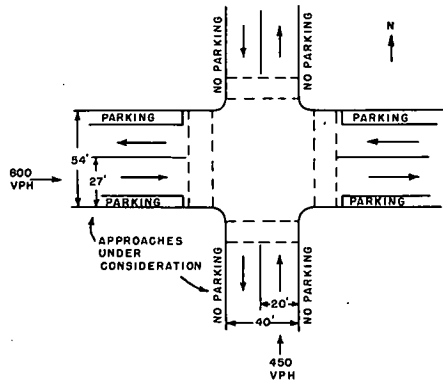
Problem:

GIVEN CONDITIONS:

East-west two-way street with parking both sides, crossing north-south two-way street with no parking; widths as shown in diagram.
Central business district.*
Metro. area population $=250,000$.*
Peak-hour factor $=0.85$.*
Signal cycle $=60$ sec, 2-phase.
Green time $=32$ sec, E-W; 22 sec, N-S.
Yellow time $=3$ sec, both streets.
Right turns $=10$ %, all approaches.*
Left turns $=10$%, all approaches.*
Trucks $=5$%, all approaches.*
Local Buses $=0$. *
Peak demand on E-W street $=800$ vph, on west approach.
Peak demand on N-S street $=450$ vph, on south approach.

DETERMINE:

(a) Load factor and intersection level of service currently existing on west and south approaches.
(b) Revised signal time split required to provide balanced levels of service on both cross streets.
(c) Load factor that currently would exist on the west approach if parking were prohibited.

_____

* Average conditions on which chart is based; consideration not necessary in this problem demonstrating another principle.

Solution:

(a) Current load factors and intersection levels of service.

WEST APPROACH

$$G/C \text{ ratio} = 32/60 = 0.53.$$

$$800/0.53 = 1,510 \text{ vphg}.$$

From Figure 6.9, for 27-ft approach width and 1,510 vphg, LF = 0.75, providing level of service E.

SOUTH APPROACH

$$G/C \text{ ratio} = 22/60 = 0.37.$$

$$450/0.37 = 1,215 \text{ vphg}.$$

From Figure 6.8, for 20-ft approach width and 1,215 vphg, LF = 0.00, providing level of service A.

(b) Revised signal time split for balanced level of service.

Trial balance, applying direct average level of service.

Average load factor = 0.38.

For west approach, from Figure 6.9, $SV = 1,380$ vphg.

$$G/C = \text{vph}/\text{vphg}$$

$$G/60 = 800/1,380$$

$$G = 35 \text{ sec}.$$

For south approach, from Figure 6.8, $SV = 1,450$ vphg.

$$G/60 = 450/1,450,$$

$$G = 19 \text{ sec}.$$

Check of new cycle

$$35 + 19 + 3 + 3 = 60 \text{ sec}; \text{ checks}.$$

Here the first trial balance proved correct. If the new cycle did not check exactly, the difference between the required and obtained cycle lengths would be prorated between the green times, and a recheck of levels of service made.

(c) Load factor on west approach under current conditions, with parking removed.

From Figure 6.8, for 27-ft approach width and 1,510 vphg, LF = 0.00, providing level of service A.

Thus, removal of parking would convert operation on the west approach from near capacity, with associated congestion, to free flow with no loading.

EXAMPLE 6.5
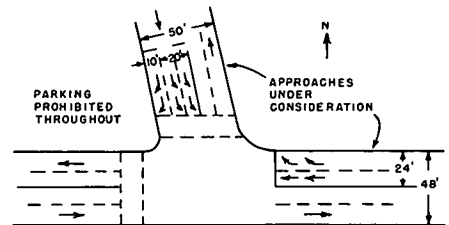
Problem:

GIVEN CONDITIONS:

T intersection, with slightly-skewed base; widths as shown in diagram.

Two-way traffic on all legs; no parking on any leg.

Offset division line on base (north) leg; approach in this leg provides two left-turn lanes and one right-turn lane.

Outlying business district.

Metro. area population = 375,000.

Peak-hour factor = 0.90.

Signal cycle = 70 sec, 2 phase.

Green time = 35 sec, north (base) leg; 29 sec, through street.

Yellow time = 3 sec, both phases.

Right turns:
N leg = 20%, on reserved lane.
E leg = 40%, on reserved lane.

Left turns: N leg = 80%, on two reserved lanes.

Trucks = 7%.

Local buses = None.

Substantial number of pedestrians in north-south crosswalk; few in east-west.

Intersection level of service C is desired.

DETERMINE:

Service volumes of north and east legs for level of service C.

Solution:

NORTH LEG

At a T intersection such as this the heavier turning movement is considered as a through movement. Because the right turns are handled by a reserved lane, "separate lane—no separate signal control" criteria apply.

Two left-turn lanes:

Figure 6.8 applies to heavy turning movement, considered here as the through movement, using 20 ft of width.

For 20-ft width and LF=0.3, $SV=1,400$ vphg.

Adjustments:

Population of 375,000 and PHF of 0.90 (Fig. 6.8) = 1.06.

Outlying business district (Fig. 6.8) = 1.25.

G/C ratio = 35/70 = 0.50.

Right turns (0% for this step) = 1.05.

Left turns (0% for this step) = 1.10.

(Note that even though there are no turns, adjustments are necessary because the "no correction" value is for 10% turns).

Trucks, 7% (Table 6.6) = 0.98.

$1,400 \times 1.06 \times 1.25 \times 0.50 \times 1.05 \times 1.10 \times 0.98 = 1,050$ vph, total service volume of two left lanes for level C.

Right-turn lane (using "separate lane—no separate signal control" criteria):

Because substantial number of north-south crosswalk pedestrians are present, use formula $600 \times G/C$ to obtain volume in passenger cars per hour, and adjust for trucks.

$600 \times \dfrac{35}{75} \times 0.98 = 294$ vph, limiting service volume of right-turn lane.

Determination of controlling volume value, with given demand distribution:

1,050 vph through = 80% of total demand volume.

$\dfrac{1,050}{0.80} = 1,312$ vph, demand volume based on through flow controlling.

Right turns = $0.20 \times 1,312 = 262$ vph, the maximum that can arrive if the through approach volume is to remain in level C, with given distribution of demand.

But 262 < 294, so is acceptable and the through flow controls.

Final service volume for north leg, level C:

$1,050 + 262 = 1,312$ vph, given 80% left and 20% right.

EAST LEG

Through lane:

Figure 6.8 applies.

For 12-ft width and LF=0.3, $SV=750$ vphg.

Adjustments:

Population of 375,000 and PHF of 0.90 (Fig. 6.8) = 1.06.

Outlying business district (Fig. 6.8) = 1.25.

G/C ratio, 29/70 = 0.41.

Right turns (0% for this step) = 1.20.

Left turns (0% for this step) = 1.30.

Trucks, 7% = 0.98.

$750 \times 1.06 \times 1.25 \times 0.41 \times 1.20 \times 1.30 \times 0.98 = 625$ vph, total service volume of through lane for level C.

Right-turn lane (referring to "separate lane—no separate signal control" criteria):

Few pedestrians present in east-west crosswalk; criteria suggest use of values for separate signal control. For level C, 800 vph applies.

$800 \times \dfrac{29}{70} \times 0.98 = 325$ vph, limiting level C service volume.

Determination of controlling volume value, with given demand distribution:

625 vph through = 60% of total demand volume.

625/0.60 = 1,040 vph, demand volume based on through flow controlling.

Right turns = $0.40 \times 1,040 = 416$ vph, maximum that can arrive if through approach volume is to remain in level C.

416 > 325, so is not acceptable.

Therefore, right-turn lane, rather than through lane, will govern.

325/0.40 = 813 vph, demand volume based on through flow controlling.

$813 \times 0.60 = 488$ vph, resulting through flow.

Final service volume for east leg, level C, = 813 vph.

EXAMPLE 6.6
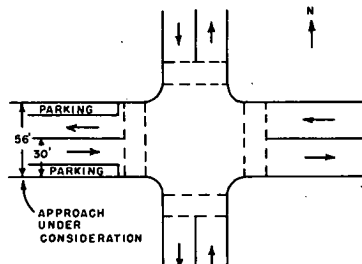
Problem:

GIVEN CONDITIONS:

Intersection of two two-way streets.

West leg approach under consideration.

Parking both sides.

Outlying business district.

Metro. area population = 100,000.

Peak-hour factor = 0.75.

Signal operation:

Total cycle length = 70 sec.

"Leading" green time with no opposing traffic; all flows in west approach move = 15 sec.

Green time with opposing traffic; all flows move = 20 sec.

"LEADING" GREEN INTERVAL   BASIC GREEN INTERVAL
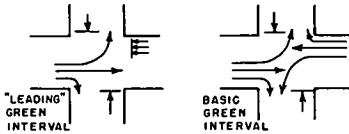
Right turns = 15%.

Left turns = 20%.

Trucks = 3%.

Local buses = None.

DETERMINE:

Service volume of west leg for level of service D.

Solution:

Figure 6.9 applies.

Load factor for level of service D, from Table 6.3 = 0.7.

For 30-ft approach width and LF = 0.7, volume = 1,700 vphg.

Adjustments for conditions not related to variations in signal indications:

Population of 100,000 and PHF = 0.75 (Fig. 6.9) = 0.85.

Outlying business district (Fig. 6.9) = 1.25.

3% Trucks (Table 6.6) = 1.02.

15% Right turns (Table 6.4) = 0.99.

$1,700 \times 0.85 \times 1.25 \times 1.02 \times 0.99 = 1,820$ vphg, chart volume adjusted for all constant values for remainder of problem.

(a) "Leading" green interval.

No opposing traffic; consider left turns as from one-way street.

Adjustments:

$G/C = 15/70 = 0.21$.

20% Left turns (Table 6.4, for left on one-way) = 0.975.

$1,820 \times 0.21 \times 0.975 = 370$ vph.

(b) Normal green interval.

Opposing traffic present.

Adjustments:

$G/C = 20/70 = 0.29$.

20% Left turns (Table 6.5) = 0.90.

$1,820 \times 0.29 \times 0.90 = 475$ vph.

(c) Total service volume of west leg

$370 + 475 = 845$ vph, at level D.

EXAMPLE 6.7
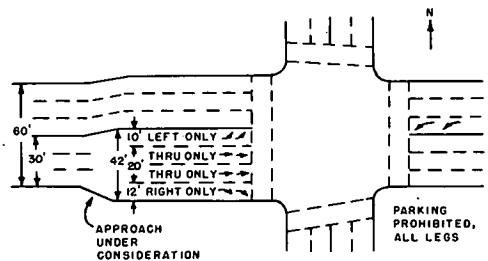
APPROACH UNDER CONSIDERATION   PARKING PROHIBITED, ALL LEGS

Problem:

GIVEN CONDITIONS:

Intersection of two two-way streets.

West leg approach under consideration; (a) it is widened from normal street width, has offset division line to provide for four approach lanes, and has lanes reserved for right turns only and for left turns only. Widths are shown in the sketch. (b) No widening.

No parking.

Outlying business district.

Metro. area population = 375,000.

Peak-hour factor = 0.85.

Signal operation:·

For Part (a) of solution:

Total cycle length = 90 sec.

Through green time = 37 sec.

Left-turn green time = 15 sec. (simultaneous with left turns from opposing direction, but separate from through time).

Right-turn green time = 55 sec. (simultaneous with left-turn time, yellow period after left time, and through time).

IS SEC   3 SEC   37 SEC

BASIC
LEFT TURN
INTERVAL
(RIGHT ALSO MOVES)

YELLOW
PERIOD
FOR
LEFT TURNS
(RIGHT ALSO MOVES)

BASIC
THROUGH
INTERVAL
(RIGHT CONTINUES)
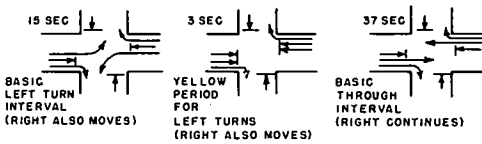
For Part (b) of solution:
  Total cycle length = 90 sec.
  Green time, all movements = 55 sec.

Right turns = 28%.
Left turns = 10%.
Trucks = 3%.
Local buses = None.
Little or no pedestrian interference.

DETERMINE:
  Volume that can be handled at level of service D:
  (a) With separate signal control for turn lanes (no conflicts with opposing flows).
  (b) Without separate signal control and without widening (30-ft midblock width maintained).
  Evaluate results.

Solution:
Figure 6.8 applies, basically, together with separate turn lane criteria.
LF = 0.7, for level D.

(a) With separate signal control for each movement.
  Through lanes:
  For 20-ft width at LF = 0.7, from Fig. 6.8, chart volume = 1,600 vphg.
  Adjustments:
    Population of 375,000 and PHF of 0.85 (Fig. 6.8) = 1.03.
    Outlying business district (Fig. 6.8) = 1.25.
    $G/C = 37/90 = 0.41$.
    Right turns (0% for this step) (Table 6.4) = 1.05.

Left turns (0% for this step) (Table 6.5) = 1.10.
3% trucks (Table 6.6) = 1.02.
$1600 \times 1.03 \times 1.25 \times 0.41 \times 1.05 \times 1.10 \times 1.02 = 995$ vph, through.

Right-turn lane (for "separate lane—separate signal control" condition):
Pedestrian interference minor.
For level D, for 12-ft lane, with 3% trucks,
  $1000 \times 12/10 \times 1.02 = 1,225$ vphg, service volume for level D.
  For 55-sec green time, $1,225 \times 55/70 = 748$ vph, service volume for level D.

Left-turn lane (for "separate lane—separate signal control" condition):
Pedestrian interference minor.
For level D, for 10-ft lane,
  $1,000 \times 1.02 = 1,020$ vphg, service volume for level D.
  For 15-sec green time, $1,020 \times 15/90 = 170$ vph, service volume for level D.

Check of relation of turn service volumes to available supply of turning vehicles, with given distribution of arriving traffic:
  Right turns plus left turns = $28 + 10 = 38\%$.
  Through volume = $100 - 38 = 62\%$.
  Through volume = $995/0.62 = 1,604$ vph, total arrival volume, level D, with given distribution of through traffic and turns, based on through volume at level D.
  Right turns:
    $1,604 \times 0.28 = 449$ vph, possible right turns arriving.
    $449 < 748$, therefore satisfactory for level D.
  Left turns:
    $1,604 \times 0.10 = 160$ vph, possible left turns arriving.
    $160 < 170$, therefore satisfactory for level D.

Final feasible level D service volume for overall approach = 1,604 vph.

(b) Ordinary single phase, without approach widening.
  Figure 6.8 applies.
  For 30-ft approach width and LF = 0.7, chart volume = 2,420 vphg.

Adjustments:
  Same as in (a), except:
    28% Right turns (Table 6.4) = 0.995.
    10% Left turns (Table 6.5) = 1.00.
    $G/C = 55/90 = 0.61$.
    $2,420 \times 1.03 \times 1.25 \times 0.61 \times 0.995 \times$
      $\times 1.00 \times 1.02 = 1,930$ vph.

Evaluation:
Results show that level D service volume is:

(a) 1,604 vph, with widening and separate turn lanes and signal intervals.

(b) 1,930 vph, without widening, and with single phase.

This demonstrates clearly that added approach lanes and multiphase operation are not automatically devices which will increase service volumes. In this case, widening together with related establishment of reserved lanes with multiphase operation on the widened roadway appears to result in a loss in effective volumes carried. The reasons for the latter result include the following:

(1) Assignment of traffic to lanes is largely proportional to distribution of demand—28% in the right lane, an average of 31% in each of the two central lanes, and 10% in the left lane—although this results in unbalanced use of available pavement.

(2) Considerable time is taken away from the through movement to accommodate the left-turn phase. Thus, at least within the approach area proper, the loss of through capability is considerably greater than the gain in left-turn capability.

(3) The left-turn lane is used mainly for storage rather than movement of traffic.

(4) The right-turn lane has considerably greater capacity than is needed for the given demand.

Some of these factors are peculiar to this particular problem, whereas others are generally associated with multiphase operation.

In this particular case, it appears that more volume might be carried if the right lane were made available for through movements as well as right turns, even though the right turns during the left-turn interval would have to be eliminated. The operation would be as follows:
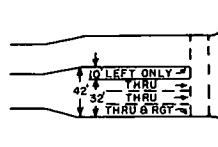
Left-turn lane:
Same as before.

Remaining lanes:
For 32-ft width at LF = 0.7, from Fig. 6.8, chart volume = 2,600 vphg.
  Adjustments:
    Population and PHF (same) = 1.03.
    Location (same) = 1.25.
    $G/C = 37/90 = 0.41$.
    Right turns, 28% (Table, Fig. 6.4) = 0.995.
    Left turns (0% for this step) (same) = 1.10.
    Trucks (same) = 1.02.
    $2,600 \times 1.03 \times 1.25 \times 0.41 \times 0.995 \times 1.10$
      $\times 1.02 = 1,530$ vph.

Total demand volume, based on through and right controlling = 1,530/0.90 = 1,700 vph, total.

Potential left turns, assuming that through and right control = $1,700 \times 0.10 = 170$ vph.

170 = 170; will probably be satisfactory, although borderline.

A demand volume of about 1,700 vph can be accommodated; 1,700 > 1,604.

EXAMPLE 6.8
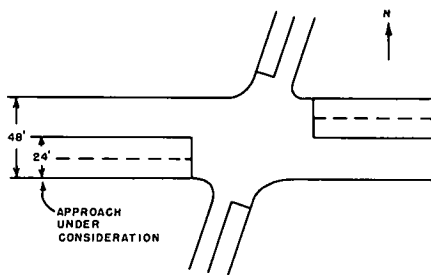
Problem:
  GIVEN CONDITIONS:
    Rural four-lane, two-way highway, at signalized intersection with two-lane, two-way highway.
    West approach under consideration.
    Average rural conditions.

Cycle length = 90 sec.
Green time = 65 sec.
Right turns = 4%.
Left turns = 3%.
Trucks = 10%.
Intercity buses = 2%.
No parking on traveled way.

DETERMINE:
Volume that can be handled at intersection level of service B.

Solution:
Figure 6.10 applies.
Load factor for intersection level B, from Table 6.3 = 0.1.
For 24-ft approach width and LF = 0.1, volume = 1,650 vphg.
For average rural conditions no adjustment for the PHF is needed.
G/C ratio = 65/90 = 0.72.
Adjustment for 3% right turns (Table 6.4) = 1.035.
Adjustment for 3% left turns (Table 6.5) = 1.07.
Adjustment for 10% trucks and 2% intercity buses (12% total) (Table 6.6) = 0.93.
$SV = 1,650 \times 0.72 \times 1.035 \times 1.07 \times 0.93 = 1,220$ vph, that can be handled at level B.

UNSIGNALIZED INTERSECTIONS

Typically, intersection capacity and signalization are closely related, because most key controlling intersections carrying heavy volumes on at least two intersecting legs are signalized. In a sense, then, capacities and the larger service volumes of unsignalized intersections may be considered of academic interest only: in practice, by the time such levels are reached at important intersections signals ordinarily will be installed. Nevertheless, in the usual urban situation many intermediate intersections will not be signalized, and in rural areas signalization is relatively rare. A brief discussion of unsignalized operation is, therefore, desirable.

An unsignalized intersection on a through route is seldom critical from a capacity standpoint. However, it may be of great significance to the capacity of a minor cross route, and it may influence the level of service on both.

In the urban case, location of the particular unsignalized intersection under consideration is of great importance in defining its capabilities. If it is relatively far from uncoordinated signalized intersections upstream and downstream, its operation may be independent of nearby signalized locations. More often, however, signal coordination causes traffic to pass an unsignalized location in a regular pattern. In one case this pattern may produce simultaneously in both directions regular gaps during which the cross street can clear, but in another may create a condition in which such simultaneous gaps seldom if ever occur. In addition, where a minor intersection is relatively close to a signalized intersection it may be influenced adversely by queues of traffic extending back from the signalized location.

In rural areas an intersection is typically a considerable distance from any other intersection and can be considered to receive traffic in random fashion. In this rural case, capacity is seldom a significant factor because operation of the through highway is usually at a sufficiently high level of service that ample gaps exist in the random pattern of traffic to accommodate typical cross traffic volumes.

Most of the limited research which has been conducted in the field of unsignalized intersections has tended to produce only locally useful results. Because of the wide possible variations in local conditions, and the consequently great difficulties involved in developing broadly applicable criteria, rigid all-inclusive unsignalized intersection service volumes and capacities cannot be presented, even for situations having apparently similar geometrics. Only generalized observations can be made.

Unsignalized intersections operate in a variety of ways, depending on the presence or absence of traffic sign controls, and the nature of these controls. Three basic categories are next discussed—no control, YIELD sign control, and STOP sign control.

No Control

It may at first seem difficult to conceive of an intersection approach operating at

capacity with no control whatsoever. Although such a condition seldom would be tolerated for an extended period, it can occur, particularly where one of the intersecting streets is decidedly inferior to the other in importance.

With no control, responsibility technically is shared equally by all drivers in making sure that the way is clear before proceeding through the intersection. In practice, however, where one highway is obviously far more important than another (as where a local county road enters a main rural highway, or where a residential street enters a heavily traveled urban arterial) the occasional cross-street vehicle yields the right-of-way to the through traffic and operation at high volumes resembles that at a two-way STOP intersection, described later.

On the other hand, where the intersecting demands are relatively balanced basic "rules of the road" govern. Although these may vary in specific localities, generally a vehicle approaching an uncontrolled intersection must yield to a vehicle approaching on the leg to its right. Again, at low volumes little delay is likely to result and capacity is not a consideration. At higher volumes performance is likely to vary widely from location to location, depending on elements such as driver characteristics and relative sight distances on the several approaches. In one case discharge from the several legs may remain well-balanced, and operation may resemble that of a four-way STOP-controlled intersection, also described later. In others, one or more legs may tend to dominate the others.

In the absence of more specific criteria, service volumes and capacities can be estimated by the previously described signalized intersection criteria, through assumption of a signalized condition in which the signal split is prorated directly on the basis of the relative volumes on the intersecting streets, and inversely on the basis of their relative widths; that is,

$$\text{Signal Split Ratio} = \frac{\text{Street 1 Time}}{\text{Street 2 Time}} =$$

$$\frac{\text{Volume, Street 1}}{\text{Volume, Street 2}} \times \frac{\text{Width, Street 2}}{\text{Width, Street 1}}$$

This is an obvious approximation in many respects, but current knowledge offers no better general alternative. The resulting values will be the maximums that could be reasonably expected; frequently influences such as non-simultaneous gaps will reduce the volumes attainable.

Yield Sign Control

The YIELD sign is primarily a tool used to establish or strengthen legal superiority of one traffic flow over another in low traffic volume locations where most traffic on all legs is able to proceed through without a full stop. From a capacity standpoint there is little, if any, difference between two-way STOP and YIELD control. Where, at a YIELD location, volumes have become sufficiently heavy that capacity must be considered, practically every vehicle on the secondary YIELD legs will be making a full stop just as under STOP control. Hence, for capacity purposes, YIELD control can be considered the same as two-way STOP control.

Stop Sign Control

The STOP sign serves two purposes—safety and facilitation of traffic movement.

The "Manual on Uniform Traffic Control Devices" (3) lists seven primary applications of STOP signs, as follows:

1. Intersection of a less important road with a main road where application of the normal right-of-way rule is unduly hazardous.

2. Intersection of a county road, city street, or township road with a state road.

3. Intersection of two main highways where no traffic signal is present.

4. Street entering a through highway or street.

5. Unsignalized intersection in a signalized area.

6. Railroad crossing where a stop is required by law or by order of the appropriate public authority.

7. Intersections where a combination of high speed, restricted view, and serious accident record indicates a need for control by the STOP sign.

Yield sign controls traffic on low-volume road crossing divided highway.

It can be seen that none of these criteria is based strictly on traffic volume, and that only the last few relate in some way to improved traffic movement. Rather, they are based primarily on safety. This is not surprising, in view of the relatively little research that has been conducted in the field of service volumes and capacities through STOP-sign-controlled intersections, and the limited local applicability of the findings.

TWO-WAY STOP CONTROL

Cross-street STOP control is normally installed to provide traffic on a through street with the right-of-way; that is, full freedom to flow without cross-traffic interference.

When the combination of the traffic on the two intersecting streets is relatively low, the intersection will work satisfactorily with no STOP signs. As traffic increases, the number of conflicts between intersecting vehicles also increases, with related increases in delay, until at some point the friction

within the intersection needs to be controlled, from both the traffic flow and the safety standpoints. This point cannot be identified by specific volume values, but it has been suggested that two-way STOP control may become desirable at the volume where 50 percent of the cross traffic is delayed beyond a normal stop because of the volume on the through street (5).

In this two-way STOP case, capacity criteria for other than the through street are relatively meaningless if the legal meaning of two-way STOP control is rigidly accepted. Very simply, in an isolated location the through-street traffic volume has complete priority over the STOP street, hence should be able to increase to capacity, while the cross-street volume gradually falls off to zero. The only feasible cross-street criteria then would be service volumes in terms of numbers of cross-street vehicles that can pass during gaps at various through-traffic service volume levels below capacity.

In practice, the problem is considerably

TABLE 6.7—EXAMPLES OF CAPACI-
TIES OF FOUR-WAY STOP INTERSEC-
TIONS WITH BALANCED DEMAND
(50-50 SPLIT OF TRAFFIC BETWEEN
INTERSECTING STREETS)

INTERSECTION TYPE	CAPACITY [a] (VPH)
2-Lane by 2-lane	1,900
2-Lane by 4-lane	2,800
4-Lane by 4-lane	3,600

[a] Total capacity, all legs.

TABLE 6.8—EXAMPLES OF CAPACI-
TIES OF A TWO-LANE BY TWO-
LANE FOUR-WAY STOP INTER-
SECTION WITH VARYING
TRAFFIC DEMAND SPLIT

DEMAND SPLIT	CAPACITY [a] (VPH)
50/50	1,900
55/45	1,800
60/40	1,700
65/35	1,600
70/30	1,550

[a] Total capacity, all legs.

more complex, involving such elements as the number of lanes on the through street, as well as on the STOP street; the availability of gaps produced by signals upstream in both directions (and the probability of occurrence of such gaps simultaneously in both directions); and the differing gap acceptance practices of drivers, depending on how long they have been delayed. If appreciable cross-street volumes exist, they may periodically "take over" the intersection, even at high through volume levels, delaying the through traffic momentarily.

Although a variety of studies of operations at two-way STOP-controlled intersections has been conducted, none appears to report directly the capabilities of cross streets at varying volume levels on the through street. Consequently, no example

is given. Were such an example available, however, it could not be applied directly to any and all two-way STOP situations, because local conditions (number of lanes and availability of gaps, particularly) vary so widely from one point to another. In short, the fact that available research findings are not specific in nature is evidence of the impracticality of establishing generalized capacities of two-way STOP-controlled intersections.

In practice, for typical computations involving the higher levels of service (lower volumes) on both streets, where most vehicles arriving at the STOP sign can enter or cross without substantial delays, the same approximate method as described for the no-control case can be used to estimate service volumes. That is, assume that a signal exists and determine, by normal intersection capacity methods, the capabilities of the assumed situation for a signal split prorated through consideration of the relative volume on the two streets and the width available to handle those volumes. Where delays become substantial this prorating is no longer valid, because unlike the no-control case one flow has legal priority over the other.

FOUR-WAY STOP CONTROL

Four-way STOP control produces more predictable traffic operation than does two-way, because all legs have equal priority. Under capacity conditions a regular discharge pattern tends to develop (often developing clockwise from leg to leg, due to the "car on right has right-of-way" rule) with very little lost time.

Studies indicate that four-way STOP control works to the best advantage of traffic when the flow on the two cross streets is approximately equal. This is probably because, no matter how unbalanced the demand may be, the regular discharge pattern tends to continue as long as there is always at least one vehicle in each approach. With unbalanced demand, consequently, the lighter flow receives unwarranted advantage over the heavier flow, in terms of discharge time received relative to demand.

It has been shown, moreover, that at low volumes a four-way STOP-sign control

can be almost as efficient as a traffic signal, and sometimes more attractive to the average driver. This is because of the flexibility it offers, compared to all but possibly the most complex forms of signal control. The driver is kept under reasonable control, but is given the opportunity to use his own judgment in deciding what movements are possible at a given instant. For instance, four right turns can be made simultaneously from four legs, whereas a simple signal would permit only two. Other combinations of several through and turning movements are possible, particularly in multilane cases.

Here again, studies of service volumes and capacities at four-way controlled intersections have been few, but in one study(6) where data are published it is concluded that total capacities of all legs combined, under balanced demand, are as given in Table 6.7. The adverse effect of unbalanced demand is given in Table 6.8 for the two-lane by two-lane crossing case.

The data in Table 6.7 generally indicate the upper limit to which four-way STOP intersections can be expected to operate.

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WEAVING

The basic components of any highway, each with its own peculiar operational features and capacity potentials, are: (1) the highway proper, characterized by uninterrupted flow (at least in the typical rural case); (2) the intersection at grade, characterized by interrupted flow (with or without signal control); and (3) the interchange, characterized by diverging or merging maneuvers. Sometimes these combine, in effect, to form yet another component referred to as a *weaving section.*

Weaving is the crossing of traffic streams moving in the same general direction, accomplished by successive merging and diverging. Thus, a simple weaving section may be described as a length of one-way roadway accommodating weaving, at one end of which two one-way roadways merge and at the other end of which they separate. In practice the approach and exit roadways may be of a variety of highway types. All may be major freeway legs, or some may be freeways and others ramps. In still other cases they may be ordinary city streets. Occasionally, weaving sections also may be found along multilane two-way roadways, but only one direction of flow will be involved in any given weave.

Many factors affect the operational characteristics of weaving sections; each must be considered in determining the capacity of, or the level of service provided in, these sections. Weaving section analysis is as necessary a part of overall highway capability determination as is analysis of any of the other components mentioned, in order to achieve a balanced design and to avoid an overestimation of the overall capacity or operating level of the highway of which the section is a part.

Regardless of the nature of the weaving section, the same operational principles and analyses in design apply, as long as fric-

tional elements are not present which prevent true weaving. Various examples illustrating the formation of weaving sections are shown in Figure 7.1. The basic weave —that is, the simple joining and subsequent division of two roadways—is shown in Figure 7.1a. Single interchanges are depicted in Figures 7.1b through 7.1e, at-grade intersections in Figures 7.1f through 7.1i, and a combination of interchanges in Figure 7.1j.

Grade-separated interchange arrangements in which weaving is inherent include the cloverleaf in Figure 7.1b, the directional interchange in Figure 7.1c, the diamond interchange with frontage roads in Figure 7.1d, and the junction in Figure 7.1e. In the case of at-grade situations, weaving is inherent in the Y-intersection in Figure 7.1f, the staggered intersections in Figures 7.1g and 7.1h, and the rotary treatment in Figure 7.1i.

Other arrangements where the weaving section is not an inherent part of any one specific interchange sometimes produce weaving sections between successive interchanges, as shown in Figure 7.1j. The closer the spacing, the greater is the influence of weaving traffic. When interchanges are spaced at sufficiently great distances the effect of weaving becomes nil, with lane changes occurring no more often than typically occurs along any section of open roadway. The section of highway may then be considered to be operating under uninterrupted-flow conditions, uninfluenced by weaving.

Weaving sections formed by successive interchanges are prevalent on urban freeways because of the need for frequent egress and ingress. Although every effort should be made in design to achieve a high operating level through greater distance between interchanges and the use of additional grade separation structures, the fact remains that

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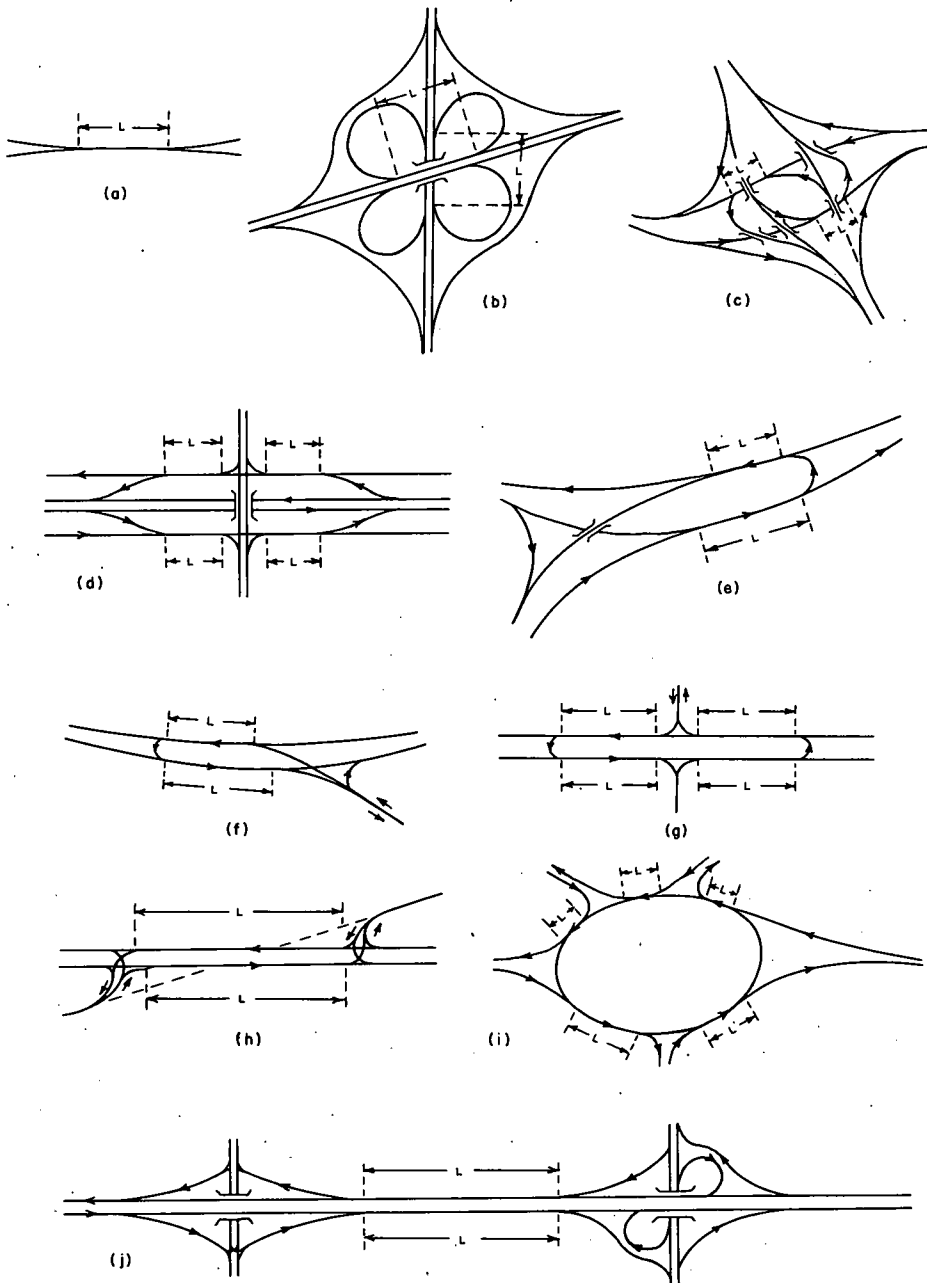
Figure 7.1. Formation of weaving sections.

many weaving sections must be incorporated in design. The need to correlate the operating levels and the capacity of weaving sections with other components of the urban freeway cannot be overemphasized.

TYPES OF WEAVING SECTIONS

Weaving sections may be considered as simple or multiple. The first involves one point of ingress and one point of egress. The second entails additional points of ingress or egress, or both. Both may be further subdivided into one-sided or two-sided sections.

Simple Weaving Sections

The various forms of simple weaving sections are shown in Figure 7.2. What may be termed as the *single-purpose* weaving section is indicated in Figure 7.2a. Here all the vehicles entering the weaving section from either approach are destined to cross the path of all vehicles entering from the other approach; that is, all traffic weaves. This is weaving in its simplest form; its application, however, is limited. One example of such operation is a weaving section between two loop ramps of a cloverleaf interchange having collector-distributor roads.

The *dual-purpose* weaving section, shown in Figure 7.2b, serves both weaving and non-weaving traffic. It is the form that is most prevalent. In this case, a sufficient number of lanes must be provided for both weaving and non-weaving traffic. Non-weaving movements, or outer flows as they are usually described, may be present on both sides, or they may be limited to one side, as in the case of a closely spaced entrance and exit on one side of a freeway.

Figure 7.2c shows the weaving maneuvers that must take place when the number of weaving vehicles is greater than can crowd into a single lane; i.e., exceed the capacity of a traffic lane. Here, some of the vehicles are involved in two weaving maneuvers. A section providing this type of operation is referred to as a *compound* weaving section. When the number of weaving vehicles is double the normal capacity of a traffic lane, four times as many weaving maneuvers must be performed as occur at one-lane capacity; these take place roughly

within three segments of the section. Theoretically, this illustrates the need to at least triple the length to accommodate twice the volume. Weaving sections on urban freeways are frequently of the compound variety.

The dual-purpose weaving section or the compound weaving section can be arranged to separate weaving traffic from non-weaving traffic; this *separated* weaving section is illustrated in Figure 7.2d. The central portion thus becomes a single-purpose weaving section as in Figure 7.2a, while the two flanking sections are devoid of weaving and carry the outer flows only. Such a separated weaving section, with only one outer roadway provided, is characteristic of an introduced section of collector-distributor road along a freeway, which thus removes weaving from the through roadway.

Multiple Weaving Sections

In contrast to the simple weaving section such as shown in Figure 7.3a, which entails a single entrance junction followed by a single exit junction, is the more complex weaving section which constitutes several ramp junctions in sequence. Such a section of highway, consisting of two or more overlapping weaving sections, is referred to as a multiple weaving section. A typical example is shown in Figure 7.3b. A multiple weaving section may also be defined as that portion of a one-way roadway which has two consecutive entrance junctions followed closely by one or more exit junctions, or one entrance junction followed closely by two or more exit junctions. Multiple weaving sections occur frequently in urban areas where there is need for distribution and collection of high concentrations of traffic. Both the operation and the analysis of multiple weaving sections are more complex than in the case of simple weaving sections.

One-Sided and Two-Sided Weaving Sections

Either of the previously described weaving section types may be further subdivided into those in which weaving takes place only on one side of the roadway and those which entail maneuvers on both sides, thus causing weaving to occur across the roadway. The two types described are shown in Figures 7.3c and 7.3d, respectively. The one-

(a) BASIC WEAVING SECTION
ALL TRAFFIC WEAVES

(b) DUAL-PURPOSE WEAVING SECTION
SERVES WEAVING AND NON-WEAVING TRAFFIC

(c) COMPOUND WEAVING SECTION
EACH WEAVING MOVEMENT INVOLVES
TWO WEAVING MANEUVERS

(d) SEPARATED WEAVING SECTION
WEAVING AND OUTER FLOWS
ON SEPARATE ROADWAYS
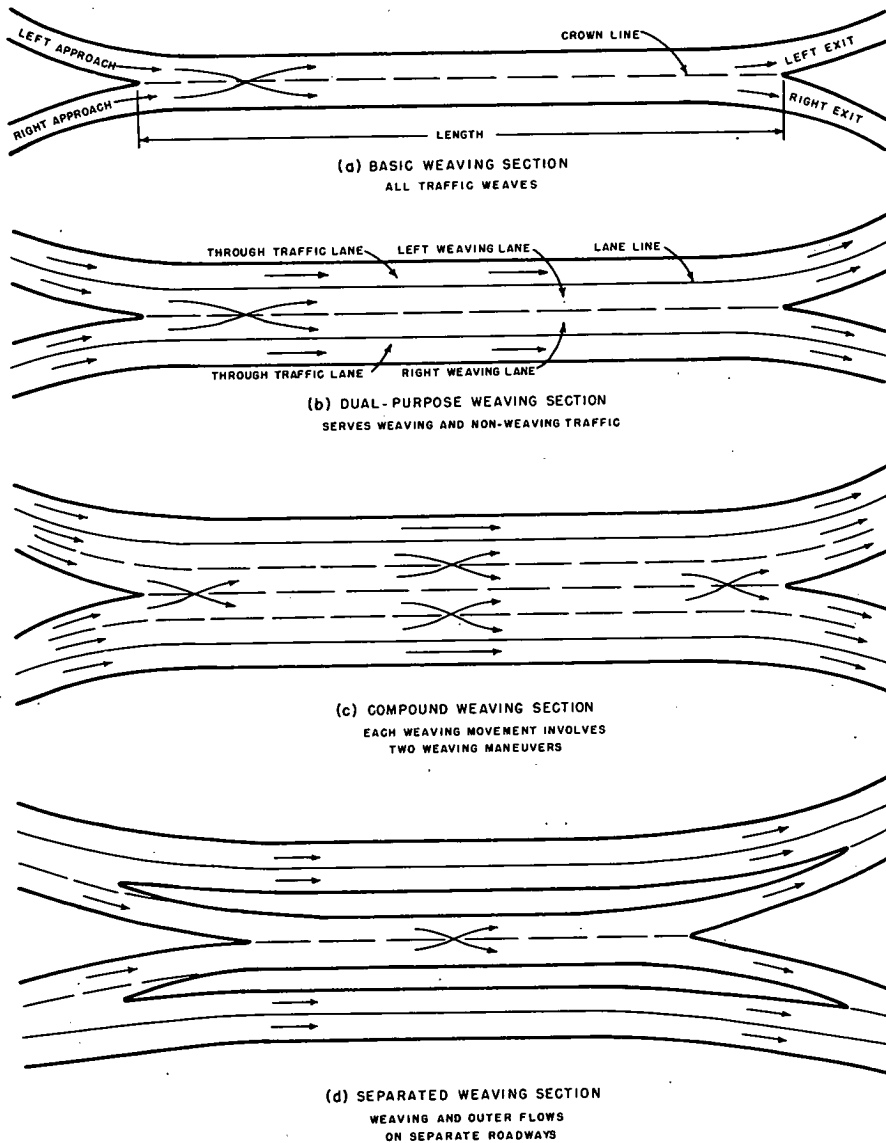
Figure 7.2. Simple weaving arrangements.

sided weaving section is typical on most freeways where the entrances and exits of interchange ramps are on the right. The two-sided weaving sections occur where the roadways of two major routes, crossing each other, are combined through a weaving section, as illustrated in Figures 7.1a, 7.1e, and 7.1h, and as shown for the rotary intersection in Figure 7.1i. Two-sided weaving sections are also likely to be found in con-

junction with major two-exit all-directional interchanges, as well as where left-hand ramps are occasionally used along with right-hand ramps.

OPERATING CHARACTERISTICS OF WEAVING SECTIONS

Weaving sections are characterized by vehicles entering a common roadway area from two or more entrance flows, and shortly afterward again splitting into two or more exit flows, within a relatively limited distance. Usually, where the several traffic flows under consideration in a given problem are of relatively balanced importance, the problem will involve fundamental weaving as described in the remainder of this chapter. Thus, basic weaving problems of the types represented by Figures 7.3a and 7.3b are usually handled by means of the methods

(a) SIMPLE WEAVING

(b) MULTIPLE WEAVING

(c) ONE-SIDED WEAVING
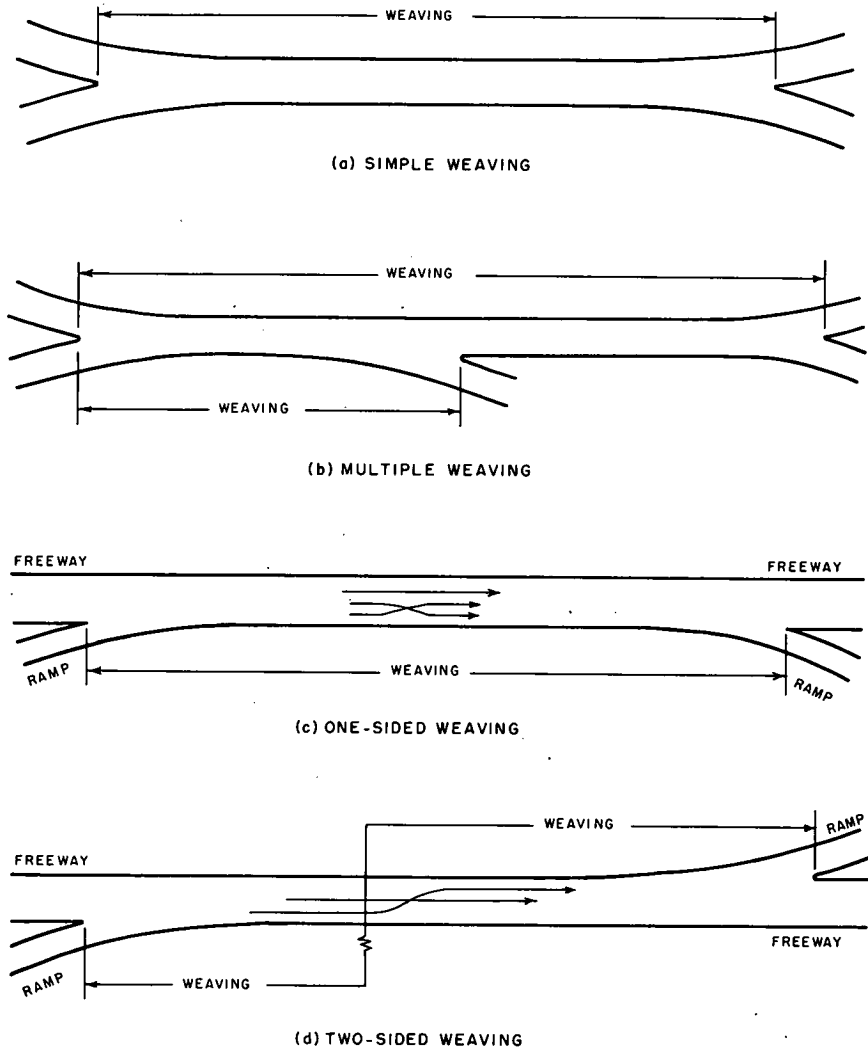
(d) TWO-SIDED WEAVING

Figure 7.3. Basic types of weaving sections.

of this chapter, particularly where major roadways are involved. The two-sided weaving represented by Figure 7.3d also is often handled by these methods. On the other hand, where the problem involves one-sided weaving, as shown in Figure 7.3c, the procedures given in Chapter Eight may be more suitable.

A weaving section handles two classes of traffic: (1) traffic entering, passing through and leaving the section without crossing the normal path of other vehicles, and (2) traffic which must cross the paths of other vehicles after entering the section. It is because of the latter group of vehicles that a weaving section is produced. On a well-designed but short weaving section operating below capacity, the two classes tend to separate themselves from each other almost as positively in practice as they do in theory. This allows each class to be examined and analyzed separately. Even at higher densities, when the two have some influence on each other due to stream friction, this general approach still may be used.

Information regarding the relations between geometric features of weaving sections and the traffic volumes and operating speeds attained on them has been obtained from detailed nationwide studies conducted by the Bureau of Public Roads (1, 2) and from other available data (3). Appendix B presents data from a group of these studies. These relationships, based on observations and operational experience, have been found to remain quite consistent over a period of years. It has been found possible to represent these fundamental weaving relationships by means of one basic weaving chart (Fig. 7.4). It includes both a graphical chart and a related formula, both of which must be utilized in any complete problem solution.

The several basic considerations included in this chart and the related formula are next discussed.

Weaving Movements

Whether all vehicles entering a weaving section are weaving vehicles, or whether the weaving vehicles are separated into their own class as discussed previously, it is obvious that every car in the weaving stream

of traffic must cross the crown line (a real or imaginary line connecting the noses of the entrance and exit forks) somewhere between its extremities (see Figs. 7.2a and 7.2b). At no instant can the number of vehicles in the act of crossing the crown line exceed the number that can crowd into a single lane. Thus, the total number of vehicles passing through a weaving section, if all must perform a weaving maneuver more or less simultaneously, cannot exceed the capacity of a single lane. It is assumed, of course, that the facility is operating as intended; that is, without vehicle movement alternating between entrance lanes as if by signal control.

In order to accommodate such weaving movements, additional roadway width beyond that on the approaches is usually required. Also, it is apparent that as the weaving volumes increase, longer distances are necessary to perform the weaving maneuvers. When the number of weaving vehicles exceeds the capacity of a traffic lane, some of the vehicles are involved in two weaving maneuvers, and compound weaving exists, as previously discussed and shown in Figure 7.2c. Where the weaving traffic approaches a volume equal to double the capacity of a traffic lane, theoretically three times as much length is required as for a weaving volume equivalent to a single-lane capacity.

The effective length of a weaving section is also influenced, at least at the better levels of service, by the distance in advance of the weaving section that drivers on one approach road can see traffic on the other approach road. This distance may be used by drivers who must cross the paths of other vehicles to adjust their speeds and position before reaching the weaving section. This effect has not yet been quantified; the curves in Figure 7.4 are based on adequate sight distances.

Weaving performance, therefore, is fundamentally dependent on the length and width of weaving section, as well as on the composition of traffic.

Non-Weaving Movements (Outer Flows)

Weaving sections normally accommodate non-weaving traffic as well, either with or without added lanes adjoining either side of the weaving lanes (Figs. 7.2b, 7.2c, 7.2d).

Figure 7.4. Operating characteristics of weaving sections.

TOTAL WEAVING TRAFFIC, $V_{we_1} + V_{we_2}$ (1,000 EQUIV. PASS. VPH)

k = 3   k = 3.0
2.9
2.8
2.6
2.4
2.2
2.0
1.8
1.6
1.4
1.2
1.0
OUT OF REALM OF WEAVING (k = 1)

I   II   III   IV

LENGTH DETERMINATION (CHART)

L = LENGTH OF WEAVING SECTION
$V_{we_1} + V_{we_2}$ = TOTAL WEAVING TRAFFIC, IN EQUIV. PCPH)
k = WEAVING INFLUENCE FACTOR
I, II, ETC. = QUALITY OF FLOW

WIDTH DETERMINATION (EQUATION)

$$N = \frac{V_{w_1} + k V_{w_2} + V_{o_1} + V_{o_2}}{SV} \quad OR \quad N = \frac{V + (k-1) V_{w_2}}{SV}$$

N = NUMBER OF LANES
V_{w_1} = LARGER WEAVING VOLUME, VPH
V_{w_2} = SMALLER WEAVING VOLUME, VPH
V_{o_1}, V_{o_2} = OUTER FLOWS, VPH
V = TOTAL VOLUME, VPH
SV = APPROPRIATE SERVICE VOLUME OR
     CAPACITY PER LANE ON APPROACH
     AND EXIT ROADWAYS
k = WEAVING INFLUENCE FACTOR

MIXED TRAFFIC,
EXCEPT IN EXTREME CASES

V_{o_1} V_{o_1}
V_{w_1} V_{w_2}
V_{w_2} V_{w_1}
V_{o_2} V_{o_2}

Figure 7.5. Method for measuring length of weaving sections.

Determination of the capacity of these non-weaving lanes involves no new principle, as they may be considered comparable to through traffic lanes on any multilane facility. If any weaving section is to function properly and efficiently, it is important that these added lanes have adequate capacity to serve most non-weaving vehicles. If non-weaving vehicles utilize the weaving lanes either through choice or through necessity, they may interfere with the vehicles that must weave to reach their destination, thus reducing the total number of weaving vehicles that can be accommodated. It may be desirable to use signs to direct drivers to the proper side of the approach road during peak periods if the number of weaving vehicles is high; however, drivers will normally use the proper lane in simple weaving sections.

Quality of Flow

Weaving section operation is measured in terms of "quality of flow." The basic chart

(Fig. 7.4) contains a family of curves designated as I through V, representing various qualities of flow ranging from excellent to poor. As discussed in detail later in this chapter, these quality of flow levels, although related to levels of service, are identical for all types of highways, whereas the corresponding levels of service vary depending on the type of highway involved.

Length Requirements

It has been shown that length of section is a significant factor in weaving. Its importance is evidenced by the fact that the fundamental weaving volume determination chart (Fig. 7.4) incorporates length as the basic variable.

The length of a weaving section is measured along the highway between the entrance and the exit, as shown in Figure 7.5. To be consistent and to comply with the data on which Figure 7.4 is based, the measurement must be made as illustrated.

In terms of the projections of the converging and diverging inner edges of traveled way of the two roadways, straight or curved as the particular roadway alinement may require, the weaving section length is measured from a point at the merging end where this distance between the projected edges is 2 ft to a point at the diverge end where the distance between the edges is 12 ft. This procedure is universally applicable to all types of junctions from single-lane ramp junctions to major forks and is equally suitable regardless of whether or not shoulders and/or curbs are present.

Where conditions permit, the adverse effect of a weaving section can be obviated, from an operational point of view, by increasing the length sufficiently between an entrance and the following exit on a highway. Further, any weaving section can be physically eliminated by the introduction of a grade separation structure. Where neither is feasible, and a weaving section remains an acceptable feature of the plan, the length of the section should be at least sufficient to provide an operating level compatible with the level of service on the highway facility of which the weaving section is a part. Operating levels are discussed in more detail later in this chapter.

Width Requirements

The basic weaving chart (Fig. 7.4) relates the weaving volumes possible at particular operating levels to the length of section only. Of equal importance, however, is another significant factor, the *width* of weaving section, in terms of the number of lanes.

In a complete solution of a weaving section problem, then, both length and width requirements must be met. This analysis involves two steps: first, determination of length based on weaving volume and desired operating level as just described, and second, determination of the width predicated on the weaving volume, the outer-flow volumes, and the lane service volume or capacity.

As mentioned previously, the weaving maneuvers and the outer flows on the shorter weaving sections tend to separate themselves into two distinct groups. The number of lanes required for the outer flows, therefore, may be calculated as for any uninterrupted

flow facility—the volume divided by the appropriate lane service volume or capacity. Thus, if the two outer flows are represented by V_{o_1} and V_{o_2}, and the lane service volume by SV, the number of lanes required to handle this traffic would be $(V_{o_1} + V_{o_2})/SV$.

The additional lanes required for the weaving movements are calculated on a parallel but not identical basis, using V_{w_1} and V_{w_2} as the two weaving volumes in the numerator and the same value of SV in the denominator. It has been shown that for equivalent volumes more width is required for weaving than for uninterrupted flow. A rational formula reflecting this fact has been developed from the available data. It states that the number of lanes required for weaving may be expressed as $(V_{w_1} + k V_{w_2})/SV$, where V_{w_1} is the larger weaving volume, in vph; V_{w_2} is the smaller weaving volume, in vph; k is a weaving influence factor, in the range of 1.0 to 3.0; and SV is the lane service volume, in vph. The k-factor, in effect, is an equivalency factor expanding the influence of the smaller weaving flow up to a maximum of three times its actual size in number of vehicles.

Combining these two expressions, and assuming that some lanes will be utilized by both outer flows and weaving movements, the complete formula for the total number of lanes in the weaving section becomes

$$N = \frac{V_{w_1} + k V_{w_2} + V_{o_1} + V_{o_2}}{SV} \quad (7.1a)$$

If $V_{w_1} + V_{w_2} + V_{o_1} + V_{o_2} = V$, the total volume of traffic accommodated by the weaving section, the equation becomes

$$N = \frac{V + (k-1) V_{w_2}}{SV} \quad (7.1b)$$

In this form the specific influence of weaving becomes clearly apparent, over and above the V/SV term representing uninterrupted flow. Both equations are included in Figure 7.4.

For convenience, a series of curves is presented for various k values. The maximum $(k = 3.0)$ is applicable to the shorter weaving sections, whose operation is represented by curves III, IV, and V. Where the actual weaving section length is greater than the minimum required, as is the case for the

Example of basic or simple weaving section, with one separated outer roadway added.

conditions to the right of curve III, the adverse influence of weaving becomes progressively less, hence the k-factor is gradually reduced, reaching a value of 1.0 for curve I. Along this curve and to the right the section is considered to be out of the realm of weaving, as discussed in more detail later, and Eq. 7.1b reduces to

$$N = V/SV \qquad (7.1c)$$

which represents the number of lanes required for uninterrupted flow under free-flow conditions.

For the better levels of service the value of SV used in the equations for determining the width of weaving section normally should be the average service volume per lane for the level of service chosen for the approach and exit roadways in question and, in the case of levels C and D, for the appropriate peak-hour factor. For freeways, these values are given in Table 9.1. They would, of course, be adjusted downward as necessary to reflect the prevailing conditions, such as

percentage of trucks, grades, and lane widths.

However, certain limitations have been established regarding the maximum value of SV to be employed in conjunction with each of the several weaving qualities of flow, I through V. These, in effect, reduce the freeway base value for lane capacity under ideal conditions of 2,000 vph, to reflect the influence of inherent weaving turbulence. These values, which still represent ideal conditions of geometrics and approach traffic, are given in Table 7.1.

In determining the number of lanes which may be required on weaving sections under heavy flow conditions, the lane service volumes of Table 7.1 should be employed, rather than basic values from Table 9.1. As before, they should be transformed to values of SV by adjustment for lane width, trucks, grades, etc.

Seldom, of course, will the resulting value of N be an even whole number; typically, it will indicate the need for a fractional part

TABLE 7.1—RELATIONSHIP BETWEEN
QUALITY OF FLOW AND MAXIMUM
LANE SERVICE VOLUMES IN
WEAVING SECTIONS

QUALITY OF FLOW CURVE	MAX LANE SV VALUE (PCPH)
I	2,000
II	1,900
III	1,800
IV	1,700
V	1,600

of a lane in addition to several full lanes. Judgment must be exercised in interpreting this result in terms of providing or not providing another lane; no arbitrary dividing line can be established. The level of service desired, as related to the size of the "leftover" fractional part, is the most important consideration. At one extreme, where (a) a high level of service (low volume) is being provided, (b) the outer flows predominate, and (c) the fractional part is small, an additional lane is unnecessary; the section can absorb the slight overload without noticeable difficulty. At the other extreme, where (a) capacity operation is involved, (b) weaving constitutes a large proportion of the total volume, and (c) the fractional part is large, an additional lane is essential.

Speed - Weaving Volume - Length - Width Relationships

Speed-volume relationships within a weaving section, coupled with the length and width of section, have a vital effect on the operating characteristics of the section and determine the quality of flow. To better understand these relationships, let it be imagined for the moment that a weaving section of very short length, say 50 to 100 ft, is under consideration. Further, let it be assumed that traffic is composed entirely of weaving vehicles. At very low traffic volumes there will be little conflict between weaving vehicles even on this short section because the entry of a vehicle from one approach will frequently coincide with a gap

in the stream of traffic from the other approach. As traffic becomes heavier, however, the probability of vehicles entering the section from the two approaches simultaneously will increase until, at moderately heavy volumes, many drivers will have to slow down to adjust their arrival times to coincide with a gap, and some will be required to stop and wait for a gap in the other stream of traffic. When the section is taxed to its capacity many vehicles will be required to come to a halt and the weaving section fails to serve its intended purpose. Operation is then comparable to that of an ordinary oblique unsignalized intersection, having a capacity of about 1,500 vph. This value corresponds to the traffic-carrying capability of a single traffic lane at rather low overall speeds, under congested flow conditions, when all vehicles have come to a stop somewhere along the approach.

Longer weaving sections, however, will carry considerably more traffic than this. If of sufficient length, they will, within reasonable limits, allow most vehicles to negotiate the weaving section without unreasonable reduction in speed. In general, the longer the weaving section, the larger the volume of weaving traffic that may be served and the greater the freedom of movement that will be achieved, provided that in all cases adequate width is provided.

Where weaving maneuvers are intensified by relatively short weaving section lengths, and a sufficient number of lanes is available for the exclusive accommodation of the outer flows, a rather positive relationship is indicated between (a) the volume of weaving traffic, $V_{wc_1} + V_{wc_2}$ (where V_{wc_1} represents the heavier and V_{wc_2} the lighter weaving movement, both in equivalent passenger cars), (b) the operating speed of the weaving traffic, and (c) the length of the weaving section, L. These relations, determined from operational experience and available data on traffic volumes and speeds on weaving sections, are shown by curves III, IV and V in Figure 7.4, the basic weaving chart, representing a choice in qualities of flow.

For conditions where drivers are obliged to separate themselves into weaving and non-weaving traffic, the three curves, III, IV, and V, are representative of perform-

ance at moderate to high volumes, at operating speeds of weaving vehicles usually approximating about 40-45, 30-35, and 20-30 mph, respectively. The curves show that for a given volume of weaving traffic, speed increases with an increase in the length of section; or for a given length of section, speed is reduced with an increase in weaving volume, to a point where the highest rate of flow—that is, capacity—is reached at optimum speed in the neighborhood of 20 to 30 mph, as shown by curve V.

Normally, capacity operation is indicative of such high demand that vehicles on the approach roadways will be in moving queues which provide a constant supply. Any further increase in approach volume or any slight mishap is likely to break down the unstable condition. With critical density exceeded, speeds fall below 20 mph, the capacity can no longer be attained, and complete congestion or stagnation, with forced flow, may occur within a few moments.

On weaving sections represented by the area to the right of curve III (Fig. 7.4) operating speeds above 40 mph may be achieved; however, either greater lengths or lower volumes than indicated by the relationship of curve III would tend to negate a positive separation of weaving and nonweaving traffic. That is, as the length of section for a given weaving volume increases beyond that specified by curve III, a progressively greater mixture of weaving and non-weaving traffic is assumed, with the result that the speed of weaving traffic, as distinct from non-weaving traffic, may or may not be greater than the 40-mph speed associated with curve III.

For this reason, no attempt is made to relate operating speeds to the curves to the right of curve III. Generally speaking, however, curve III represents good operation with only slight speed adjustments required by weaving vehicles, provided that adequate width is available. Similarly, curve I represents a smooth flow condition where the speed of weaving traffic would approximate the speed of all traffic in the section as determined by the total volume on the roadway under the prevailing traffic and roadway conditions, as on any uninter-
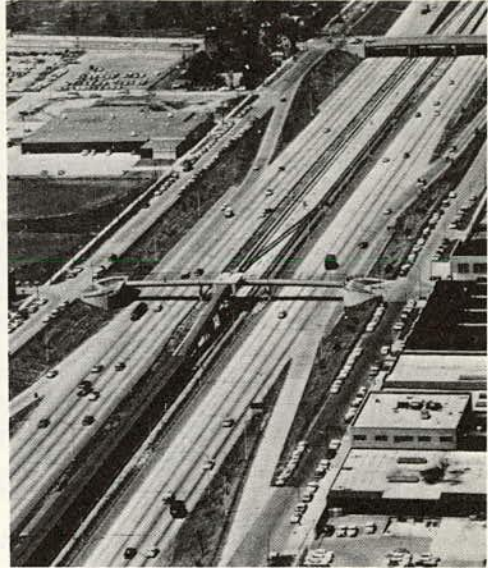
Freeway weaving area between interchange ramps.

rupted-flow facility; typically, this curve would apply to low-volume, high level of service operation.

On the average, on freeways, operating speeds through weaving sections for a given level of service will fall from 5 to 10 mph below those for the same level on the roadway sections forming the entrances and exits.

It will be noted that in the area between curves III and I, length is substituted for width to a limited extent. That is, the longer the section for a given weaving volume, the lower will be the value of the k-factor substituted in the equation to determine the number of lanes required.

Sections out of Realm of Weaving

The effect of weaving is intensified as the length of weaving section is reduced. Conversely, the effect of weaving is lessened as the length of weaving section is increased. Therefore, there must be a point in length where the weaving maneuvers are strung out over such a long distance that the effect of weaving, as such, is nullified or dissipated. Thus, where the distance between an entrance and an exit along a roadway is so

Cloverleaf interchanges include inherent weaving sections.

great that the effect of weaving maneuvers is no more than that of normal lane changing, the section ceases to function as a regular weaving section.

Data are insufficient to state definitely the circumstances for which the effect of weaving may be considered dissipated and, therefore, the conditions or lengths for which it is not necessary to design a roadway as a weaving section. There is some indication that, beyond certain lengths and within certain weaving volume limits, operational levels of capacity are little affected by weaving.

From this it has been rationalized that when the volume-length combinations given in Table 7.2 obtain, or the lengths given are exceeded, it is not necessary to design or evaluate the operation of a roadway section on the basis of weaving section criteria.

The relations of Table 7.2 actually form the basis for curve I in Figure 7.4. Values which would fall on or to the right of this curve are considered to be out of realm of weaving and are representative of uninterrupted flow conditions. Values which fall above and to the left of curve I are taken to represent a weaving condition. Those between curves I and III, as previously discussed, are indicative of excellent to good operating conditions in the weaving section, provided, of course, an adequate number of lanes is furnished.

TABLE 7.2—VOLUME-LENGTH COMBINATIONS CONSIDERED OUT OF REALM OF WEAVING

VOLUMES WEAVING [a] (PCPH)	MIN LENGTH OF SECTION, L (FT)
500	1,000
1,000	2,400
1,500	4,000
2,000	6,000

[a] $V_{we_1} + V_{we_2}$

LEVELS OF SERVICE AND CAPACITY

It has been pointed out that weaving may occur under a wide variety of conditions on highways of all types, ranging from freeways to ordinary urban streets. The weaving criteria contained in this chapter, represented by Figure 7.4, have been developed for broad application to pure weaving situa-

TABLE 7.3—RELATIONSHIPS BETWEEN BASIC ROADWAY LEVELS OF SERVICE AND QUALITY OF FLOW ON WEAVING SECTIONS

LEVEL OF SERVICE	QUALITY OF FLOW [a]			
	FREEWAYS AND MULTILANE RURAL HIGHWAYS		TWO-LANE RURAL HIGHWAYS	URBAN AND SUBURBAN ARTERIALS
	HIGHWAY PROPER	CONNECTING COLLECTOR-DISTRIBUTOR ROADS AND OTHER INTERCHANGE ROADWAYS		
A	I-II	II-III	II	III-IV
B	II	III	II-III	III-IV
C	II-III	III-IV	III	IV
D	III-IV	IV	IV	IV
E [b]	IV-V	V	V	V
F	←———————— Unsatisfactory [c] ————————→			

[a] As represented by curves of Figure 7.4. Relationships below heavy line not normally considered in design. Where two entries are given, that on the left is desirable, that on the right is minimum.
[b] Capacity operation.
[c] Maximum volume equivalent to V, but may be much lower.

tions on highways of any of these types. Because each of these types has its own individual scale of levels of service, it is not possible to apply the basic level-of-service designations A through F to the curves on the weaving chart (Fig. 7.4) on an all-inclusive basis. Rather, those curves are considered to represent several levels of quality of flow, designated by I through V. Table 7.3 serves as a cross-reference relating these quality designations to the equivalent levels of service on the particular highway type under consideration.

In Figure 7.4 the various curves (I through V) relate weaving volume to length of weaving section. Each curve represents a certain combination of relative operating conditions or intensity of weaving. Curve I, for instance, designates an operating condition where the weaving vehicles have little if any effect on the quality of traffic flow. At the other extreme, curve V is indicative of the most intense weaving conditions, representing capacity or the maximum hourly rates of flow which can be achieved for a given length of weaving section. The intervening curves (II through IV) are indicative of the intermediate range of conditions.

Inasmuch as weaving sections introduce added stream friction, it can be expected that operating speeds through them will be somewhat more restricted than on the open road, at any given volume level. In any case, they should provide a capacity equal to or greater than that of the approaching roadways; otherwise, the restrictive effect of weaving can be reflected over a substantial length of the through roadways. Although it is desirable to maintain the same (equivalent) level of service (i.e., quality of flow) throughout the highway section as a whole, achievement of this result is often impracticable because of physical limitations. Where this is the case it should be recognized that a lower quality of flow will result in the weaving section, as compared to the highway's overall level of service.

In this connection it should be pointed out that operation through a weaving section is different from operation along an open roadway section of the same highway in that a greater percentage of traffic than normal is changing lanes, at somewhat lower speeds. An increase in driver tension may be experienced, with varying degrees of
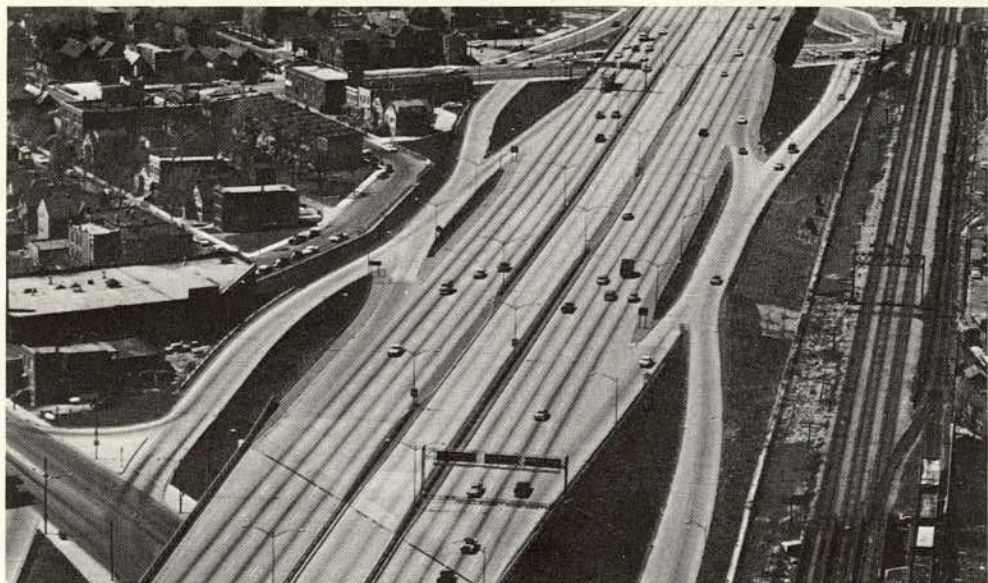
Collector-distributor roads alongside a freeway incorporate weaving sections.

driver satisfaction with the overall operation. Recognizing this, the engineer should strive for minimum loss in the quality of traffic flow through a weaving section. To the greatest extent feasible, therefore, the operating level of a weaving section should be made compatible with the level of service on the highway of which the weaving section is a part. However, provided the level is not much lower than on the highway as a whole, and provided this lower level is not found at frequent intervals along the route, it may prove reasonably acceptable to most drivers.

The quality of operation representative of the several weaving qualities may be described as follows:

I. Operating conditions and speeds approach those normally found under free-flow conditions without weaving, being determined largely by average lane volumes. The effect of weaving on stream flow is slight, if any. Thus, with the appropriate number of lanes speeds of 50 mph or more are feasible.

II. Operating conditions and speeds are only slightly more restricted than those generally found under free-flow conditions without weaving. The effect of weaving on stream flow is slight to nominal. Some speed variations will occur, but with an appropriate number of lanes operation at about 45-50 mph can be achieved.

III. Weaving vehicles can maintain operating speeds in the order of 40-45 mph, although speed will vary considerably between individual vehicles and between short periods within the hour. Non-weaving vehicles can maintain higher speeds if sufficient capacity has been provided in non-weaving lanes. Drivers are affected by other vehicles in the stream to a greater extent than normal under free-flow conditions, but the level of operation is not unreasonable for the condition where operating speeds on the approaches are 50 mph.

IV. Although speeds will vary considerably between individual vehicles, weaving vehicles can maintain operating speeds of about 30-35 mph. Non-weaving vehicles can main-

tain higher speeds if sufficient capacity has been provided in the outer lanes. Occasional slowdowns and some restriction of maneuverability can be expected, but operation is acceptable to drivers where approach speeds typically do not exceed 40 mph, especially in highly developed areas or where short-trip traffic predominates on the roadway.

V. This represents capacity for a given length of weaving section—that is, the maximum number of vehicles that can be accommodated in 1 hr. At such capacity flow

Examples of two-sided, dual-purpose weaving sections on freeway segments.

the speed may be quite variable, normally below 30 mph and frequently averaging 20 mph or even less. Slow operation and turbulence, including stopping of weaving vehicles, alternating of weaving movements between lanes, and nosing into the parallel lane by drivers in one weaving lane, are common occurrences. Non-weaving vehicles may or may not move reasonably well through the section proper, depending on the capacity provided in the non-weaving lanes. Minor accidents may be expected at a fairly high frequency. Usually, backup and loss of service are evident on at least one and possibly both approach legs during high-flow periods, affecting the non-weaving traffic as well as that which will weave. This type of operation is not acceptable for design purposes.

The design of a weaving section should be based on the general level of service intended for the entire highway of which the weaving section is a part. In this regard, considerable judgment must be exercised. Table 7.3 cross-references the quality of operation in a weaving section, as represented by curves I to V of Figure 7.4, that is considered compatible with the basic levels of service on any particular highway type of which the weaving section may be a part. The relationships below the heavy line normally are not considered in design. For each level of service the first operating level, where two are shown, is considered to be the desirable value and the second is taken normally to be the minimum value for design of weaving sections. The desirable values should be striven for in design of weaving sections adjoining freeway-to-freeway interchanges and for two-sided weaving sections on freeways.

PROCEDURES FOR DESIGN AND OPERATIONAL EVALUATION OF WEAVING SECTIONS

Simple Weaving Sections

GENERAL CONSIDERATIONS

Direct analysis of simple weaving sections is relatively easy, involving use of the basic weaving chart and equation in Figure 7.4 together with reference to Tables 7.1 and 7.3, to determine length and width of section, given demand volumes. The traffic flows through the weaving section must be shown in their separate components, including outer flows, larger weaving flow, and smaller weaving flow, as defined in Figure 7.4. The procedures can also be used in reverse when geometrics are given and operating characteristics are required.

The curves of Figure 7.4 link the three basic weaving factors—length of weaving section (as defined in Fig. 7.5), total weaving traffic, and quality of flow. Knowing any two of these makes it possible to find the third. In the typical problem, level of service rather than quality of flow will be known, hence the quality of flow in the weaving section must be correlated with the levels of service for each type of highway, by means of the relationships shown in Table 7.3. It is important to note that the weaving volumes shown on the vertical scale of the chart are expressed in terms of equivalent passenger cars per hour.

The equation in Figure 7.4 relates the several traffic flows through the section, level of service, and number of lanes required. In the equation all traffic volumes may be expressed in vehicles per hour of mixed traffic, unless grades or truck percentages vary widely between the several legs. In such cases conversion to equivalent passenger cars, or development of a composite service volume value reflecting average conditions, must be considered.

The level of service desired controls the value of SV, the appropriate service volume or capacity for the type of highway of which the weaving section is a part, determined by the methods described in Chapters Nine and Ten. As usually used in weaving problems, it is an average of the service volumes of the several freeway legs involved.

Several cautions are necessary regarding selection of the appropriate value of SV, in addition to consideration of widely differing grades and truck volumes as just mentioned. First, in determining the average value of SV for the several approach and exit roadways it is important to remember that in the case of freeways the service volume per lane on the approach and exit roadways will vary, for a particular level of service, depending on the number of lanes,

as discussed in Chapter Nine. Second, the maximum value limitations on SV for the several qualities of flow, as presented in Table 7.1, must be taken into account. Finally, in the case of one-lane collector-distributor roads, specific service volume criteria have not been established. However, a review of Table 7.3 indicates that the per-lane volume limits for the next poorer level of service than that under consideration for the overall problem are suitable. Use of the values in Table 9.1 for two lanes in one direction on four-lane freeways, divided by two, is suggested.

As previously discussed, rounding off of the fractional part of a lane usually included in the result, N, requires consideration of level of service, extent of weaving, and relative size of the remainder.

The typical examples which follow show the several ways in which the procedures may be applied, depending on which variables are given quantities and which are required.

TYPICAL PROBLEM SOLUTIONS—SIMPLE WEAVING

EXAMPLE 7.1

Problem:
What are the minimum length and width requirements for the weaving section formed by the intersection of highways AB and CD (see diagram) to give level of service B, under the demand volume conditions shown in the figure? The highways accommodate rural resort traffic with a negligible volume of trucks. Traffic lanes are 12 ft wide, clearances are adequate, and grades are level.

(TRAFFIC VOLUMES IN VPH)
PLAN

WEAVING DIAGRAM
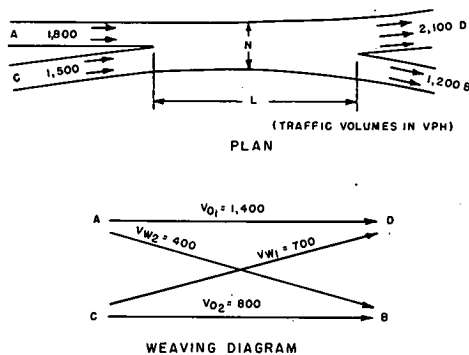
Solution:
Values required for use in entering Figure 7.4:

The volume of truck traffic is negligible, hence no conversion is required to express weaving volumes in terms of equivalent passenger cars per hour, either for use of the basic chart or for the equation. Considering first the approach roadway, it is noted that the specified ideal geometrics require no adjustments for clearance or grade. Referring to Table 9.1, for level of service B under these ideal conditions, the maximum service volume of 2,000 passenger cars per hour listed for a two-lane one-directional roadway is found to approximate the approach and exit roadway conditions. Comparison of this 2,000 value with the given demand volumes on the approach and exit legs shows that, with one borderline exception, they can be accommodated at the specified level. Therefore, the average service volume per lane on approach and exit roadways is established as 2,000/2, or 1,000 equivalent passenger cars per hour. A check of Table 7.1 shows that this 1,000 value is acceptable for use in the width equation.

From Table 7.3, for level B on a basic freeway section, quality of flow is II. From Figure 7.4, $k=2.6$ for quality of flow II. Then $V=$ total volume $=1,800+1,500=$ 3,300 vph and $V_{we_1}+V_{we_2}=700+400=$ 1,100 pcph (trucks negligible, so conversion not made).

Length of weaving section:
Entering Figure 7.4 with the total weaving volume of 1,100 pcph, and projecting horizontally to the quality II curve, then projecting vertically, the length required is read as 1,500 ft.

Width of weaving section (number of lanes):

$$N = \frac{V + (k-1)V_{w_2}}{SV}$$

$$= \frac{3,300 + (2.6 - 1)(400)}{1,000}$$

$$= 3.9,$$

which is so close to 4 that no "rounding-off" question exists.

Result:

The conditions require a 1,500-ft weaving section with four lanes. (Also, consideration should be given to the use of three lanes on exit D, which is slightly outside level B with only two lanes. All the other legs are adequate with two lanes.)

EXAMPLE 7.2

Problem:

The diagram shows the peak-hour traffic volumes projected for a weaving section on a proposed urban freeway. Trucks constitute 3 percent of the traffic. The freeway is being designed with 12-ft lanes and 10-ft continuous shoulders and the weaving sec-

(TRAFFIC VOLUMES IN VPH)

PLAN

WEAVING DIAGRAM
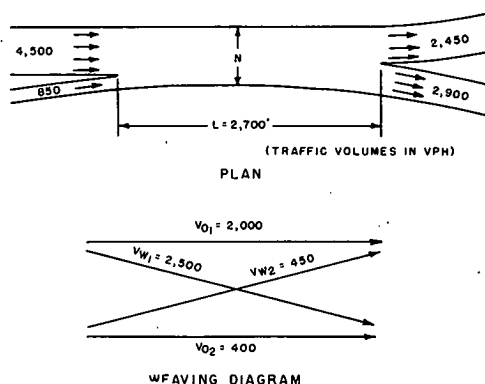
tion under consideration is centered in a 1-mile-long 3 percent upgrade. The desired level of service on the freeway is C and the peak-hour factor is 0.91. The maximum length available for weaving is 2,700 ft. What will be the value of k, the level of service, and the approximate speed of weaving traffic? How many lanes will be required? Does this proposed design provide the desired level of service?

Solution:

Equivalent weaving passenger cars:

The 3 percent grade requires that, for use of the chart in Figure 7.4, the weaving volumes be converted to equivalent passenger cars per hour. (Conversion is not necessary for the numerator of the width equation). The truck adjustment factor, $(100 - P + E_T\,P_T)/100$ (where P_T is the percentage of trucks and E_T is the appropriate passenger car equivalent; see Chapter

Five), is the reciprocal of the appropriate tabulated truck factor in Table 9.6. For the purposes of this problem, the portions of the grade influencing operations would be the ¼ mile upstream of the section (the first ¼ mile of the grade) plus the ½-mile length of section, or ¾ mile total effective length. For a 3 percent grade ¾ mile long with 3 percent trucks, the passenger car equivalent from Table 9.4 would be 10, and the truck factor, T_L, from Table 9.6 would be 0.79.

Then $V_{we_1} + V_{we_2} = (V_{w_1} + V_{w_2})/T_L = (450 + 2,500)/0.79 = 3,735$ pcph.

Width of section (number of lanes), k factor, and quality of flow:

The available length of weaving section is given as 2,700 ft. With both the weaving volume and the length known, Figure 7.4 is used to determine the quality of flow and the k factor, both identified by the intercepts of the projections of the two known values. The quality of flow is found to lie between III and IV, with $k = 3.0$. Inspection of Table 9.1 for level of service C as related to the given demand volumes indicates that the average number of lanes on the approach and exit legs will be 3. (In most actual problems these numbers of lanes would be known from separate analyses of these roadways.) Hence, the maximum service volume under ideal conditions for PHF = 0.91 is taken as 4,350/3 or 1,450 passenger cars per hour per lane. No adjustment is required here for lane width or lateral clearances because these geometrics are ideal; the truck factor must be applied, however, in determining SV. Therefore, the service volume on approach and exit roadways is $SV = MSV\ (T_L) = 1,450\ (0.79) = 1,145$ vph. A check of Table 7.1 shows this value to be acceptable for quality of flow between III and IV. Also,

$$N = \frac{V + (k-1)\,V_{w_2}}{SV}$$

$$= \frac{(2,000+400+2,500+450) + (3.0-1)\,(450)}{1,145}$$

$$= 5.5.$$

Here, for intermediate level C with weaving predominating, this should be rounded upward to 6 lanes.

Level of service and approximate speed in section:

Referring to Table 7.3, the level of service for the prevailing quality of flow in the weaving section between III and IV, is D. The text indicates that the approximate speed of weaving traffic is about 35 mph. Then $k = 3.0$, 6 lanes are required, the level of service is D rather than C as desired, and the approximate speed is 35 mph.

Evaluation of results:

For level of service C, operating speeds of 50 mph would exist on the approach and exit roadways, and corresponding speeds through the weaving section would typically be in the 40-45 mph range. That is, the 35 mph here obtained confirms the finding of level D rather than the desired level C operation; that is, operation below usual design levels.

Attainment of level C is unlikely within the length limitation; a length of at least 3,700 ft would appear necessary to achieve it. Recomputation using level D criteria would show that five lanes would be adequate to provide service within level D, commensurate with that possible within the given length limitation.

EXAMPLE 7.3

Problem:

A diamond-type interchange is to be located on a freeway downstream of but close to a fully directional, freeway-to-freeway interchange. For the traffic movements shown in the diagram, what distance is required between the merging end and the exit nose to take the intervening freeway section out of the realm of weaving? A level of service of B is to be maintained on the freeway, which has 12-ft lanes and relatively level grades. Truck traffic is estimated to be 5 percent with a passenger car equivalent of 2. How many lanes are required between interchanges?

Solution:

From Table 9.6, for 5 percent trucks with $E_T = 2$, the truck adjustment factor is $1/T_L = 1/0.95 = 1.05$.

For the freeway to be out of the realm of weaving, $k = 1.0$ where $V_{we_1} + V_{we_2} = (800 + 550) \times 1.05 = 1,420$ pcph.

(TRAFFIC VOLUMES IN VPH)

PLAN

WEAVING DIAGRAM
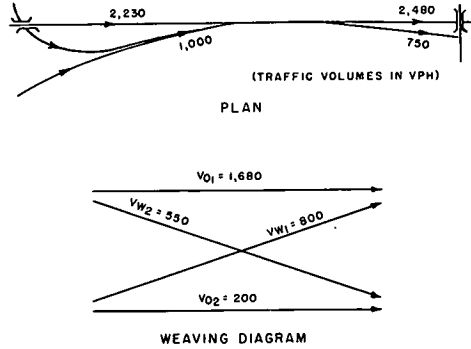
From Figure 7.4, the length of the section required for $k = 1.0$ is 3,800 ft.

The volume through the section is $V = 1,680 + 200 + 800 + 550 = 3,230$ vph, and the number of lanes is determined simply by V/SV, because conditions are outside the realm of weaving.

Here, an approximate choice of N must first be made, inasmuch as SV on a freeway is dependent on the number of lanes. Reference to Figure 9.1, for level B, indicates that under ideal conditions 3 lanes would be adequate, handling 3,500 pcph. Assume 3 lanes for computing SV per lane, or $SV = (3,500/3)(T_L) = 1,166(0.95) = 1,110$ vph per lane. This is acceptable, according to Table 7.1, and $N = V/SV = 3,230/1,100 = 2.91$, or use 3 lanes.

Note: If the assumed number of lanes differs from that as computed in the foregoing, check the effect of recomputing SV per lane on the basis of the revised number of lanes.

EXAMPLE 7.4

Problem:

Peak-hour traffic volumes for a proposed cloverleaf intersection on a rural freeway are shown in the diagram. Determine whether or not a separate collector-distributor road would be justified on the basis of length of weaving section and number of lanes required. The level of service on the freeway is C, and the peak-hour factor is 0.91. Trucks comprise 6 percent of the traffic, the freeway has 12-ft lanes and fully continuous 10-ft shoulders, and the grades on the through lanes are approximately 2 percent down.

WITHOUT C-D ROAD

(TRAFFIC VOLUMES IN VPH)

PLAN

WEAVING DIAGRAM

WITH C-D ROAD

PLAN

WEAVING DIAGRAM
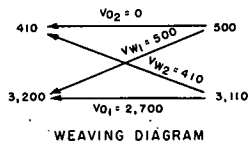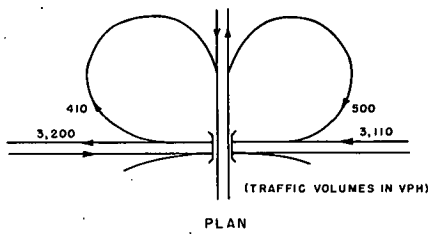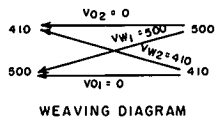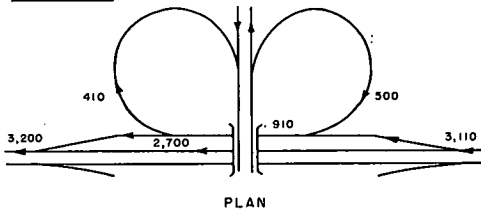
Solution:

On a 2 percent downgrade, the passenger car equivalent is the same as on level ground. From Table 9.4, $E_T = 3$.

For 6 percent trucks, the truck adjustment from Table 9.6 = 0.89.

Then $V_{wc_1} + V_{wc_2} = (500 + 410)/0.89 = 1,020$ pcph.

The level of service is C. Using Table 7.3, the desirable quality of flow is II on the freeway and III on the C-D road, for which the k values are 2.6 and 3.0, respectively.

(a) Without C-D road:

The desirable quality of flow = II, and $k = 2.6$.

Using Figure 7.4, the length of weaving section required is 1,400 ft.

Using Table 9.1, for a level of service C on the freeway with 0.91 PHF, the average per-lane ideal service volume for the prob-

able 3-lane freeway legs and 1-lane ramp legs is that for 2 lanes (2,750/2, or 1,375 passenger cars per hour).

$SV = 1,375 \ (T_L) = 1,375 \ (0.89) = 1,225$ vph per lane; acceptable according to Table 7.1.

$V = 2,700 + 0 + 500 + 410 = 3,610$ vph.

$$N = \frac{V + (k-1) V_{w_2}}{SV}$$

$$= \frac{3,610 + (1.6 \times 410)}{1,225}$$

$$= 3.5.$$

Use four lanes.

(b) With separated *C-D* road:

The desirable quality of flow = III, and $k = 3.0$, on the C-D road.

Using Figure 7.4, the length of weaving section required is 550 ft.

The volume on the C-D road = 910 vph.

The level C service volume for approach and exit lanes, considered applicable to these C-D road conditions, can be taken from Table 9.1 as approximately one-half of the 2-lane value, or 2,750/2 = 1,375 vph.

$SV = 1,375 \ (0.89) = 1,225$ vph on the C-D road; acceptable according to Table 7.1.

The number of lanes required on the C-D road $= (V_{w_1} + 3 V_{w_2})/SV = [500 + (3.0 \times 410)]/1,225 = 1.4$.

Use 2 lanes.

The number of through lanes required on the freeway = $V/SV = 2,700/1,225 = 2.2$.

Use 3 lanes, as would definitely be required on the approach and exit portions of the freeway.

Although a total of 5 lanes is required through the weaving area in the C-D road case, as compared to 4 lanes without the C-D road, the C-D road appears to be the more practical solution if a cloverleaf interchange is to be used; that is, a weaving length of 1,400 ft is not practicable between the loop ramps, whereas a length of 550 ft is near the usual maximum dimension achieved on well-designed cloverleafs.

Multiple Weaving Sections

GENERAL CONSIDERATIONS

Because multiple weaving sections are made up of overlapping simple weaving sections, each analyzed separately, the basic relationships in Figure 7.4 and the related tables apply to the analysis of multiple weaving sections as well. Some of the varieties of multiple weaving sections are shown in Figure 7.6, in which a simple section also is shown for comparison. These types include a single entrance followed by (a) two exits and (b) three exits; two successive entrances followed by (c) one exit and (d) two exits; or three successive entrances followed by (e) one exit. Obviously, other combinations are also possible. Ramps may be all on the right or there may be a mixture of right-hand and left-hand ramps.

A multiple weaving section is referred to as having two, three, or more segments, where a segment is that portion of a multiple weaving section between any two successive ramp junctions. The weaving sections in Figure 7.6a and 7.6b are two-segment and three-segment varieties, respectively. Each segment is analyzed individually with regard to length and width requirements. For example, in Figure 7.6a the weaving maneuvers on the longer weaving section (between points 1 and 3) are divided into two portions—those that weave in Segment 1, and the remainder that weave in Segment 2. Then, the weaving movements in the shorter weaving section (Segment 1) include not only the essential direct weaves between points 1 and 2 but also the portion of the weaves in the longer weaving section (point 1 to point 3) occurring in Segment 1.

The manner in which weaving traffic divides itself between the various segments of a multiple weaving section can only be estimated. Considerable variation occurs, depending on geometrics, truck traffic, signing, and other factors. For purposes of analyses, it is considered reasonable to assume that weaving along the longer sections is propor-
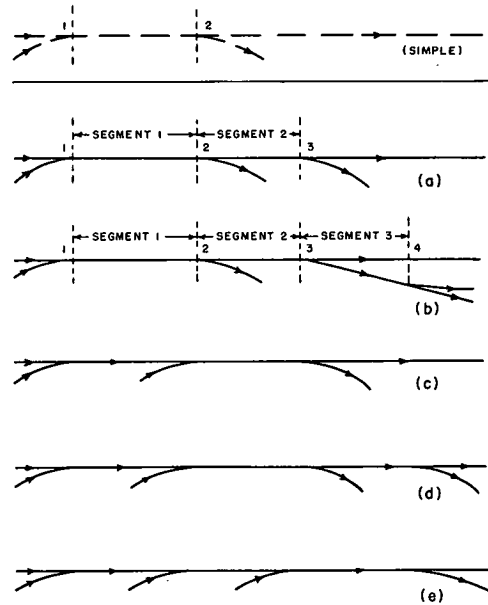
Figure 7.6. Types of multiple weaving sections.

tional to the lengths of segments within these sections, and thus to allocate the weaving on that basis.

The chart in Figure 7.4, then, is used in a manner similar to that for simple sections to establish the length requirements for each segment, using the summation of all the weaving movements within the segment, still on an equivalent passenger car basis. The required number of lanes for each segment is also determined individually in accordance with the basic formula, modified as follows:

$$N = \frac{V + \Sigma(k-1)V_{w_2}}{SV} \qquad (7.2)$$

The summation sign, Σ, is introduced ahead of the second term in the numerator to account for the more than one set of weaving movements in some of the segments. Inasmuch as each such set of weaving movements may have a different value of k, this form of summation is essential.

Multiple weaving requires other special considerations in connection with the alloca-

tion of weaving to two or more segments of the weaving section in proportion to their lengths. In setting up the weaving diagram, care must be taken to identify and separate the various weaving movements (primary and secondary weaves) to avoid double-counting. This principle is discussed and illustrated in Example 7.5, in which a movement already handled in full in one step is omitted in subsequent calculations. It is again applied in Example 7.6.

A problem may arise in the use of the basic formula (Eq. 7.2) for determining the number of lanes when the omitted volume happens to be the smaller of the two secondary weaving movements. In this case the unduplicated (larger) weaving volume is substituted for V_{w_2} in the equation. This procedure also is illustrated in Example 7.6.

On a well-designed freeway multiple weaving sections normally will not exceed three segments. Nevertheless, the procedure outlined applies to weaving sections with any number of segments. However, it should be recognized that the analysis becomes more complex as the number of segments increases. For this reason, any one analysis is limited normally to three segments. For more than three segments the analysis would be made in two separate parts overlapping each other. For example, a four-segment weaving section would be analyzed for the first three segments as if it were a complete weaving section, then the last two segments (third and fourth) would be analyzed separately in like manner. Inasmuch as the third segment is analyzed in both parts, the more critical of the two results for this segment would be used in design.

Detailed handling of the multiple weaving method is best described by the following actual examples.

TYPICAL PROBLEM SOLUTIONS—MULTIPLE WEAVING

EXAMPLE 7.5

Problem:

Determine the number of lanes required for the freeway multiple weaving section shown in the diagram for level of service C with PHF=0.91 on the freeway. State whether the lengths of weaving segments

OVERALL WEAVING DIAGRAM
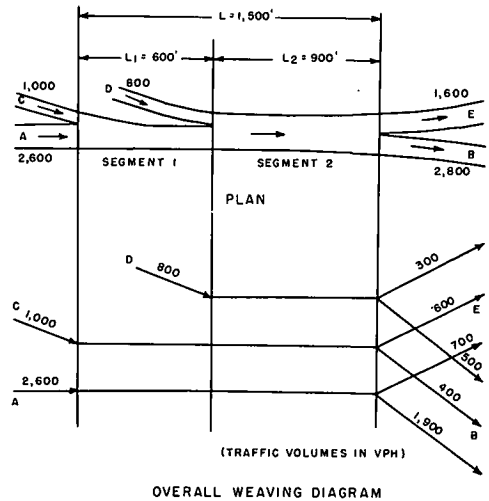
shown to be available are adequate to provide the desired service. The proportion of trucks, having passenger car equivalent of 3, is 8 percent. Assume that the average service volume for the given conditions (level of service, PHF, and trucks) has already been determined as 1,200 vehicles per hour per lane.

Solution:

Weaving between movements CB and AE is proportional to L_1 and L_2; thus,

Segment 1:
    Movement CB = $400 \times (6/15) = 160$ vph
    Movement AE = $700 \times (6/15) = 280$ vph
Segment 2:
    Movement CB = $400 - 160 = 240$ vph
    Movement AE = $700 - 280 = 420$ vph

INCREMENTAL WEAVING
DIAGRAMS
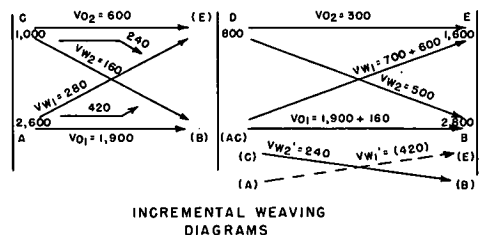
Note: In Segment 2, the secondary 420-vph portion of weaving movement AE, which did not weave in Segment 1, shown as the dashed line, has already been included in the

total 700-vph AE weaving movement within movement (AC)-E, all of which weaves with DB. Therefore, in subsequent calculations for length and number of lanes, this 420-vph volume is omitted.

For level of service C, Table 7.3 gives the minimum quality of flow as III and the desirable as II.

From Table 9.6 for passenger car equivalent of 3 and 8 percent trucks, T_L, the truck adjustment factor = 0.86.

SEGMENT 1

$$V_{we_1} + V_{we_2} = \frac{1}{T_L}(V_{w_1} + V_{w_2}) = (1/0.86)$$

$(280 + 160) = 510$ pcph.

From Figure 7.4, for service quality III the required minimum length is 200 ft, and for service quality II the desirable length is 650 ft. Thus, the available length of 600 ft is adequate for purposes of weaving.

For length 600 ft, $k = 2.8$. A check of Table 7.1 shows the specified service volume of 1,200 vph per lane to be acceptable.

Number of lanes $N = \dfrac{V + (k-1)V_{w_2}}{SV}$

$= [3,600 + (1.8 \times 160)]/1,200 = 3.2.$

Use 4 lanes.

SEGMENT 2

$$(V_{we_1} + V_{we_2}) + V_{we_2}' = \frac{1}{T_L}[(V_{w_1} + V_{w_2}) + V_{w_2}'] = \frac{1}{0.86}[(1,300 + 500) + 240] = 2,370$$

pcph.

From Figure 7.4, for quality III the required minimum length = 1,900 ft. The available length of 900 ft is inadequate even as a minimum for the level desired. It will provide only quality of flow IV, with operating speeds likely to be 30 mph or less. The level of service available through Segment 2, then, is below that provided in the remainder of the freeway. The section is a

potential bottleneck because further volume increases will cause it to reach capacity before the remainder does. For length 900 ft, from Figure 7.4, for $V_{we_1} + V_{we_2} = \dfrac{1}{0.86}$ $(1,300 + 500) = 2,090$ pcph, $k = 3.0$. Similarly, for $V_{we_1}' + V_{we_2}' = \dfrac{1}{0.86}(420 + 240)$ $= 765$ pcph, $k = 2.7$.

Number of lanes

$$N = \frac{V + \Sigma(k-1)V_{w_2}}{SV}$$

$$= \frac{4,400 + (2.0 \times 500 + 1.7 \times 240)}{1,200}$$

$$= 4.8$$

Use 5 lanes.

Note: The numbers of lanes for the several approach and exit roadways are determined by the methods in Chapter Nine, following which the overall layout is reviewed for balance and flexibility, and to insure that rigid adherence to computed results has not produced unreasonably frequent changes in number of lanes.

EXAMPLE 7.6

Problem:

For the freeway multiple weaving section shown in the diagram, determine the number of lanes required in each segment and establish whether or not the length of each weaving segment is compatible with a level of service C on the freeway, given a peak-hour factor of 0.83. Freeway geometrics are ideal, and the proportion of trucks, with a passenger car equivalent of 3, is 5 percent. For these conditions, a service volume of 1,200 vehicles per hour per lane has been established.

L1 = 1,700' L2 = 570'

SEGMENT I   SEGMENT 2

L = 2,270'

PLAN

A 2,600

910

(TRAFFIC VOLUMES IN VPH)

OVERALL WEAVING DIAGRAM
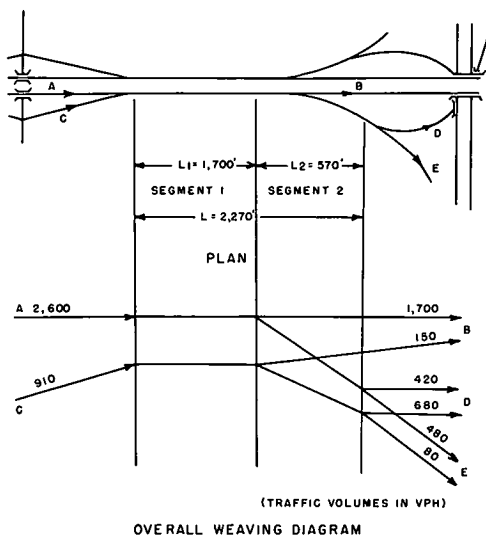
Solution:

Weaving between AE and CD is proportional to L_1 and L_2; thus,

INCREMENTAL WEAVING DIAGRAM
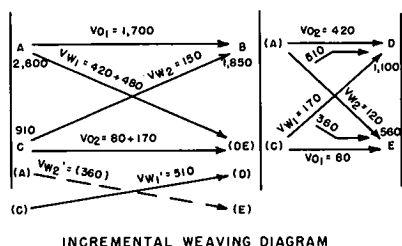
Segment 1:

Movement $CD = 680 \times (1,700/2,270)$
$= 510$ vph
Movement $AE = 480 \times (1,700/2,270)$
$= 360$ vph

Segment 2:
Movement $CD = 680 - 510 = 170$ vph
Movement $AE = 480 - 360 = 120$ vph

Note: See Example 7.5 for discussion of movement represented by dashed line.

For level of service C, from Table 7.3 the quality of flow desirably is II, and from Table 9.6 for passenger car equivalent of 3 and 5 percent trucks, the truck adjustment factor is $T_L = 0.91$.

SEGMENT 1

$$(V_{we_1} + V_{we_2}) + V_{we_1}' = \frac{1}{T_L}[(V_{w_1} + V_{w_2}) +$$

$$V_{w_1}'] = \frac{1}{0.91}[(900 + 150) + 510] = 1,715 \text{ pcph}.$$

From Figure 7.4 for quality II, the required length is 2,600 ft, which exceeds the available 1,700 ft. However, Table 7.3 indicates that quality III is an acceptable minimum level; this requires only 1,070 ft. Therefore, the length of 1,700 ft is reasonably adequate for purposes of weaving.

For length 1,700 ft, from Figure 7.4, for $V_{we_1} + V_{we_2} = (1/0.91)(900 + 150) = 1,155$ pcph, $k = 2.4$. Similarly for $V_{we_1} + V_{we_2} = (1/0.91)(510 + 360) = 955$ pcph, $k = 1.8$. SV is acceptable, according to Table 7.1.

$$N = \frac{V + \Sigma(k-1)V_{w_2}^*}{SV}$$

$$= \frac{3,510 + (1.4 \times 150 + 0.8 \times 510^*)}{1,200}$$

$$= 3.4.$$

Use 4 lanes.

SEGMENT 2

$V_{we_1} + V_{we_2} = (1/T_L)(V_{w_1} + V_{w_2}) = (1/0.91)$
$(170 + 120) = 320$.

From Figure 7.4 for quality III, the required minimum length is 100 ft and the desirable quality III length is 400 ft. The available length of 570 ft is therefore adequate for purposes of weaving.

For length 570 ft, $k = 1.2$.

$$N = \frac{V + (k-1)V_{w_2}}{SV}$$

$$= \frac{1,660 + (0.2 \times 120)}{1,200}$$

$$= 1.4.$$

Use 2 lanes.

Each weaving segment is longer than the minimum required for level of service C. Therefore, the whole weaving section is compatible with the remainder of the freeway.

---

* V_{w_2}' is not included in the calculations for length and number of lanes because it is already included in the main weaving movement. In the calculation for N, V_{w_1}' is therefore substituted, it being the lowest real number.

Weaving Under Other-Than-Freeway Conditions

GENERAL CONSIDERATIONS

Thus far, this chapter has concentrated principally on weaving as found on freeways, where marginal frictions are largely absent. Often, however, operations on more ordinary highway types also involve weaving in varying degrees. Such operation sometimes entails many adverse influences, which may prevent the high-caliber weaving associated with freeways.

On ordinary highways other than freeways in rural areas, as long as reasonably free flow is achieved, the procedures described for freeways remain generally valid. This applies to weaving along the highway between reasonably closely spaced junctions, as well as to weaving within well-designed rotary intersections of adequate dimensions. On the other hand, where certain design features of low standard are prevalent within the weaving section care should be exercised to account for these inadequacies. Usually this is accomplished by appropriate reduction in the lane service volume or capacity, SV, in determining the number of lanes.

On major streets in urban areas, disturbing elements within the weaving section (such as signals, driveways, exits and entrances to business establishments, pedestrians, parking, or vehicles stopping to pick up or discharge passengers) can place severe limitations on the use of weaving sections. Although the individual effects of these factors cannot be evaluated directly, their influence should be recognized in design and in analysis of operation. First, the value of SV which determines the width should be appropriately reduced to reflect these influences, as on any highway. Second, the effect of adverse conditions should be further recognized through use of judgment in choosing the proper curve of Figure 7.4 for determining the length; i.e., for any intended operating level a length greater than that represented in the chart should be selected. For example, if an operating level represented by curve V is expected where frictional elements are known to be present, the length chosen might be taken from curve
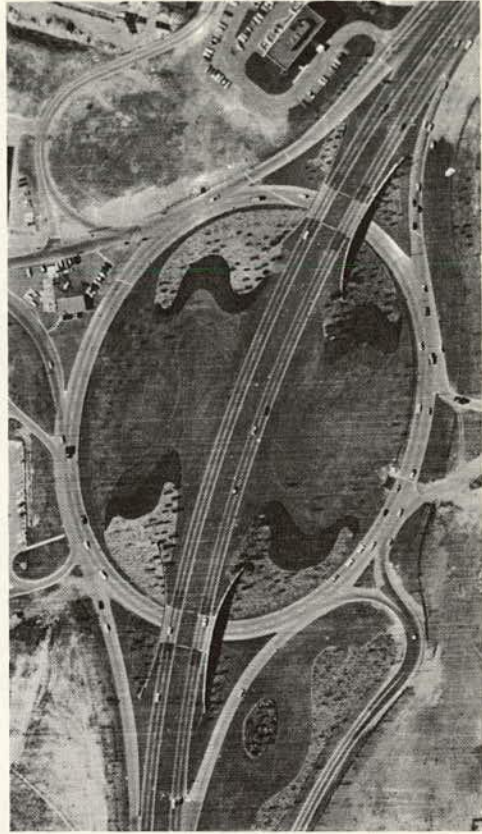
This rotary includes a series of weaving sections; it serves as a collector road for freeway and local traffic.

IV; or, if an operating level equivalent to IV is desired the length might be selected between curves IV and III. Thus, the additional length would tend to compensate for the adverse conditions described. No rigid rules or criteria can be given for such cases; good engineering judgment must be applied in solving such problems. In some cases, such as where a rotary of very limited dimensions is involved or where traffic signals within or near the weaving section prevent free weaving, it may prove impossible to analyze the location as a weaving section.

An example follows, to illustrate the method.

TYPICAL PROBLEM SOLUTION—WEAVING UNDER OTHER-THAN-FREEWAY CONDITIONS

EXAMPLE 7.7

Problem:

The elongated rotary shown in the diagram is located on an urban arterial street without control of access, with parking permitted. There is commercial development on the periphery of the rotary, involving pedestrian movements and local traffic movements to and from commercial driveways and entrances. The proportion of trucks in the traffic flow is approximately 10 percent, with an assumed passenger car equivalent of 2. Under these several restrictive conditions, a service volume of 900 passenger cars per hour per lane has been established as feasible, based on the approach and exiting roadways.

For the peak-hour traffic volumes shown, find both the desirable and the minimum length of and the number of lanes required for the section marked "A".

PLAN

WEAVING DIAGRAM
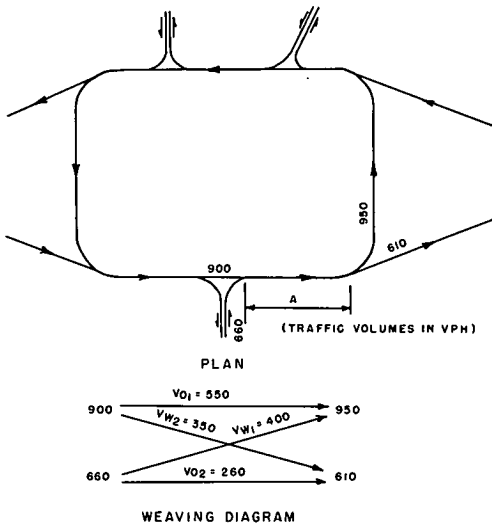
Solution:

The effect on traffic flow of pedestrians, parking, and unlimited entering and exit movements, and other roadside frictional effects is to reduce sharply the service volume of the through street as compared to the service volumes attainable on freeways. As covered in Chapter Ten, the effect of these several frictional effects cannot be evaluated directly, and to some extent is a matter of experience and judgment. In this problem, which is illustrative only of application of a service volume once obtained, the figure of 900 passenger cars per hour per lane has been assumed as a reasonable value, and found acceptable by Table 7.1. In actual problems, analysis of the entire urban street system composing the rotary and its approaches would be necessary to develop the appropriate value.

From Table 10.6 for $E_T = 2$ and 10 percent trucks, the truck adjustment factor $= 0.91$.

$V_{wc_1} + V_{wc_2} = (1/0.91)(400 + 350) = 825$ pcph.

In Table 7.3 the desirable quality of flow is III for a desirable level of service A and IV for a minimum level of service D.

Using Figure 7.4,

(a) for both cases, $k = 3.0$.

(b) for desirable level of service A, the length of weaving section required $= 380$ ft.

(c) for minimum level of service D, the length of weaving section required $= 120$ ft.

SV is given as 900 pcph, therefore $SV = 900 (0.91) = 820$ vph.

$$N = \frac{V + (k-1)V_{w_2}}{SV}$$
$$= \frac{1{,}560 + (3.0 - 1.0)\,350}{820}$$
$$= 2.8.$$

Use 3 lanes for moving traffic, in addition to any parking lanes required. (*Caution:* Parking lanes must be 12 to 14 ft wide, in average cases, to assure minimum influence on moving traffic. The service volume given for use in this problem presumably reflects the influence of parking in the particular problem at hand.)

REFERENCES

1. NORMANN, O. K., "Operation of Weaving Areas." *HRB Bull. 167*, pp. 38-41 (1957).
2. HESS, J. W., *Traffic Operation in Urban Weaving Areas.* Bureau of Public Roads (1963 data, in preparation for publication).
3. LEISCH, J. E. Unpublished studies (1958-64).

CHAPTER EIGHT

RAMPS

INTRODUCTION

A ramp is a roadway that permits traffic to transfer from one highway to another. It is broadly defined as an interconnecting roadway of a traffic interchange, or any connection between highways at different levels or between parallel highways, on which vehicles may enter or leave a designated roadway. More specifically, for the purposes of this chapter, a length of roadway is assumed to exist between terminals; that is, an opening in a narrow outer separator could not be analyzed accurately by means of the procedures contained herein.

The following discussion of ramp operations and influences may be applied to all multilane highways having ramp connections, but because ramps are used primarily with freeways, this type of roadway is normally referred to in the text.

The efficiency of traffic movement on the through lanes of a freeway may be directly affected by the adequacy of the associated ramps. Inadequate entrance ramps can seriously limit the volume of traffic that can enter a freeway, with congestion resulting on the through roadway lanes if the limiting volume is exceeded. Inadequate exit ramps also can cause freeway congestion, due either to the basic inability of an exit ramp to accommodate traffic leaving the freeway or to a backup from other restrictive conditions farther along the ramp, such as insufficient provision for traffic discharge into the neighboring street system. The proper design and placement of ramps on high-volume highways is, therefore, imperative if those highways are to offer fast, efficient, and safe operation.

The development of such suitable designs to provide for given volumes of traffic depends to a great extent on the determination of ramp and ramp junction capacities. Although the expression "ramp capacity" is frequently used, actually it is only in relatively special cases that the capacity of the ramp proper (that is, the turning roadway) governs the amount of traffic carried. In the great majority of cases, merging and diverging conditions at the ramp termini control the volume carried. Hence, this chapter is concerned primarily with determination of ramp junction service volumes and capacities, rather than those of the ramp roadway.

Closely related to junction performance is another important factor, the performance of freeway lane 1, the right-hand through lane in the ramp terminal area. Because the volume in freeway lane 1 will often control the ramp service volumes and capacity, close consideration also is given to procedures for the determination of lane 1 volumes at critical locations under various conditions.

The capacity and service volume determination procedures here presented are based on the analysis of a large number of ramp junction studies conducted throughout the nation, and are intended for both operational and design applications.

They combine two recently developed service volume determination procedures into a workable whole (1, 2, 3). Although either of these procedures could be utilized in solving certain problems, the Committee has elected to specify those levels of service for which each is considered most appropriate. Both procedures are based on substantial amounts of actual field data, but, because much still remains to be learned about ramp and ramp terminal operations, there remain certain designs and special problems not yet adequately covered. Study and understanding of the principles set forth in Chapters Seven and Eight should, however, assist the engineer in solving these special problems; approximate methods of handling several are described. References to

several other studies in this field are included at the end of the chapter (*4, 5, 6, 7, 8*).

GENERAL CONSIDERATIONS

The following are some of the more important factors which relate to design of ramps and to traffic operations in connection therewith. These should be kept in mind throughout any evaluation of the performance of an existing ramp and should be similarly considered during the design of any new ramp-freeway junction. It should be recognized that considerable reduction in level of service under high traffic flow conditions can be expected if these factors are disregarded.

Weaving Between Ramps

Where an on-ramp junction is followed a relatively short distance downstream by an off-ramp junction, a case of one-sided weaving, as described in Chapter Seven, usually exists. Research has shown the volume distributions by lane at certain critical points in the merging and diverging sections to be significant. Also important in some cases is the rate at which weaving occurs. Allowance for these factors in design through application of specified service volume criteria at these selected checkpoints and specified maximum weaving rates, helps to assure provision of an acceptable level of service for the type of weaving found between on- and off-ramps. Auxiliary lane sections between ramps also involve consideration of these service volume checkpoints and weaving limitations.

Weaving between major roadways with two or more lanes on all branches is not covered in this chapter; reference should be made to Chapter Seven for such weaving. Generally, Chapter Seven is directed toward weaving between the main lines of two or more freeways, whereas the procedures in this chapter are more appropriate for most weaving between successive ramps, with or without auxiliary lanes, on one side of a freeway.

Consideration of Peak-Period Volumes

For no portion of a highway is it more important to know the volume of traffic in various movements during peak periods than

for ramp junctions. Such knowledge is equally essential whether the problem involves the design of an adequate ramp or the analysis of an existing ramp. Hourly volume data may be inadequate. The peak flow on one ramp may occur at a different time within the hour than the peak flow on the freeway served.

In applying the procedures contained in this chapter to the solution of specific existing operational problems, then, peaking characteristics within the hour may be found critical, in which case a rate of flow based on a period of time shorter than one hour should be used. For design applications, on the other hand, the volumes used will normally be the estimated design hour demand for the future design year. This chapter includes discussion regarding application of the peak-hour factor to ramp junction problems.

Influence of Design

The design of the entrance and exit terminals of ramps appears, by general observation, to be an obvious factor influencing overall junction operations. Designs which have sharp curvature adjacent to the freeway, poor sight distance, inadequate length for accomplishing merging, diverging, or speed-change functions, or poor delineation of vehicle paths, should be avoided, because they tend to produce erratic operation. Terminal designs on new facilities should provide easy, natural pathways with adequate sight distance and good delineation. Interchange patterns should remain as simple and as similar in operation as possible, consistent with need and economy. Standardized exit and entrance terminal designs, as proposed by a number of investigators and designers, are being adopted in some areas in the field.

Nevertheless, many below-standard termini currently exist in the field, on highways built some years ago or fitted into very limited rights-of-way. Detailed analyses of the many field studies on which the procedures in this chapter are based have, unfortunately, failed to provide positive quantitative indications of the effect on ramp junction capacities of inadequacies such as those previously listed. Paradoxically, field studies frequently have shown that ramp-freeway terminals of substandard design on older, overloaded free-

ways have actually carried volumes comparable to those carried at terminals having high design standards, where sufficient demand existed. In any case, level of service is considerably poorer at the substandard locations, with a resultant penalty to both the ramp and the freeway stream, either upstream or downstream of the terminal area.

The specific adverse effects of various design inadequacies, therefore, cannot be presented on the basis of data currently available; they can be discussed only in general terms. The computation procedures that follow later in this chapter for the several levels of service assume reasonably modern and adequate designs. Research is currently under way to provide more specific knowledge of the effect of absence of acceleration lanes. Adjustment factors for application to such older designs may result from this work.

Driver experience is important in interchange operation. Poor use of ramps may be observed where unfamiliarity or lack of experience play an important role. These effects frequently appear to overshadow those of design variations. Generally, the most efficient operation is found in large cities; these larger urban areas, which have had considerable freeway mileage open for relatively long periods of time, have a greater proportion of experienced freeway drivers than do smaller cities in which freeways have been more recently introduced. Similarly, interchanges carrying predominantly commuter traffic tend to have smoother operating characteristics than those carrying the same volume of tourist or long-distance traffic.

Downtown area freeway, showing ramps to and from cross streets.

Factors Controlling Ramp Capacity

GENERAL

The overall capacity of a ramp is the least of three values: (1) the capacity of the terminal at the ramp-freeway junction, (2) the capacity of the ramp proper, and (3) the capacity of the terminal at the junction with the surrounding street system. Where two freeways intersect, of course, all ramp termini will be ramp-freeway junctions. Single-lane entrance and exit ramps will

largely prevail, being utilized wherever practicable.

Many terminals serving the surrounding street system are, in effect, at-grade intersections, and capacity is calculated in accordance with the rules for calculating intersection capacity (see Chapter Six). It frequently will be found necessary to widen a typical one-lane exit ramp to two or even three lanes at its junction with a cross street in order to provide sufficient lanes on the approach to the signal to achieve a balance

between the capacity of the ramp roadway and the ramp terminal, and to avoid a back-up onto the freeway. Seldom, however, is there a need to provide a full-length two-lane ramp.

The capacity of a single-lane ramp turning roadway between termini can, under ideal conditions, reach 2,000 vehicles per hour, or the same as a through roadway lane, if the single-lane portion is short. Restricted geometrics (grades, curvature, and the like) on most actual ramps, however, result in considerably lower values in most cases. Inasmuch as terminal capacities seldom reach this "ramp proper" value unless a lane is added to the freeway upstream of the exit terminal or downstream of the entrance terminal, such volumes are seldom seen, and one lane is generally adequate.

Two-lane ramps may be required, however, where ramp volumes exceed the capacity of the desired service volume level on single-lane entrances and exits. Also, if a high-volume ramp roadway is more than 1,000 ft long, or if it is on an upgrade and handles appreciable volumes of trucks, two lanes are necessary in order to provide for passing, breaking up of queues, and filling of large gaps, which will permit higher speeds and a more even rate of arrival at the merging terminal. A two-lane ramp may be tapered to one lane at the freeway terminal provided the flow rate does not exceed 1,500 vph, or 30 veh per minute, over any peak 5-min period. If the volume exceeds this amount, the ramp should not be squeezed down to one lane, but a long parallel auxiliary lane should be provided, subject to capacity checks described in the later portions of this chapter.

At the ramp-freeway terminal, the volume of traffic in lane 1 of the freeway has a marked effect on merging and diverging operations and will usually control the ramp service volumes and capacity that can be achieved.

ENTRANCE RAMP JUNCTIONS

At entrance ramp junctions, with only isolated exceptions, the ramp vehicle driver has the task of evaluating the freeway stream and making speed adjustments necessary for merging into a chosen gap. Some limited

assistance by lane 1 vehicle drivers may be given in that some of these drivers may speed up or slow down to widen gaps, crowd over toward the left edge of lane 1 while in the merging area, or even move into adjacent lane 2. Often such shifts into lane 2 will not be apparent to the ramp driver because the move takes place somewhere upstream of the entrance ramp in anticipation of conflict in the merge area.

Occasionally, vehicles on a heavily-used entrance ramp having excellent geometrics will, in effect, control the merge, with lane 1 vehicles tending to yield more than is customary. Usually the freeway volumes are light at such locations and the merge will consist predominantly of ramp vehicles.

The critical element in evaluation of entrance ramp capabilities is the availability of sufficient time-space in the lane 1 traffic stream. The cumulative time-space during a short interval is essentially more important than the size and distribution of individual headways or gaps, because drivers in both streams will make adjustments to allow individual vehicles to enter provided there is enough total time for both streams. In other words, the essential factor in ramp service volume and capacity determination is the ability to estimate lane 1 volume at merging areas, given the freeway and ramp volumes, and distances to and volumes on adjacent ramps.

EXIT RAMP JUNCTIONS

In the case of exit ramps, estimation of lane 1 volume immediately upstream of the exit is the essential factor in ramp service volume and capacity determinations, because the volume in lane 1 at such locations will have a considerable effect on the level of service provided by the freeway. The problem areas associated with exit ramps can be separated into three categories, as follows: (1) prevention of volume overloading of lane 1 of the freeway immediately upstream from the exit, (2) provision of an efficient design for the exit ramp and its speed change area, and (3) provision of adequate design and capacity at the connecting street terminal of the ramp. Although these three categories are interrelated to some degree, the emphasis in this chapter is on the first mentioned, the determination of the

volume loading of lane 1 immediately upstream from the exit. This volume will have a considerable effect on the speed and quality of operation found in the vicinity of the exit ramp. The remaining factors, involving design, also will have an effect on the speed and quality of operation, but research has not yet answered all the questions relating to these elements. It is significant to note that investigators making operational studies frequently observe geometric elements that appear to have been adopted without proper consideration of the results of previously published research. Certainly, an essential need is for a design which will provide adequate distance for deceleration of exiting vehicles *after* their divergence from the through lane 1 of the freeway. It is important, too, that the driver be able to perceive that the necessary speed reduction can be accomplished after divergence. Ideally, then, the speed of lane 1 vehicles upstream from an exit ramp should be dependent on volume, rather than on geometrics and signing, as is often the case.

Depressed urban freeway section with diamond ramps leading to and from one-way frontage roads.

LEVELS OF SERVICE AT RAMP TERMINALS

Because ramps are the important input-output elements of the freeway, the emphasis in this chapter is on estimating volumes which will result in an acceptable level of freeway flow in ramp terminal areas.

The remainder of the chapter is concerned primarily with the description and computation of service volumes and capacity at freeway terminals of entrance ramps and exit ramps and along the intervening freeway roadway where an entrance ramp is followed by an exit ramp a short distance downstream.

Levels of service, as previously mentioned, are qualitative measures of the effect of a number of factors, including the degree of driver satisfaction afforded. On through roadway sections, operating speeds are one measure of this factor throughout the range of operating levels. Speeds also have been used as the measure through "major" weaving areas. At ramp junctions, however, the situation is more complex. On the freeway lanes, speed remains a relatively good measure provided it is recognized that drivers have somewhat less freedom to maneuver in

the vicinity of ramp terminals than they would on through sections, just as is the case in weaving sections. Characteristics of the merge and diverge operations are such that approximate level of service criteria can be assigned to them.

On the ramp turning roadway itself, the situation is different and not yet fully understood. At present, insufficient facts are available to permit establishment of detailed level of service criteria on the ramp proper. However, it is doubtful that rigid volume criteria could be established no matter how many data were available, due to the greater significance of the merge and diverge conditions.

All sections of this manual dealing with service volumes and capacities of specific highway elements, other than basic long sections of highway, attempt to specify conditions that will be in harmony with the chosen levels of service on the through highway itself. Thus, the operating levels for ramp terminal conditions must relate to the volume found in lane 1 of the freeway in the vicinity of ramp terminals, if such balanced condi-
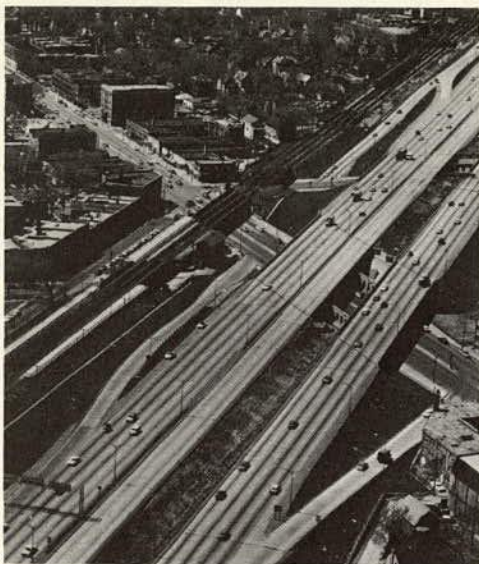
Use of two successive entrance ramps without an exit ramp between.

tions are to be maintained. It is the overall terminal condition, rather than the separate conditions in lane 1 and on the ramp, that is considered in detail in the discussion and procedures that follow. Overall freeway levels of service, over substantial lengths, are covered in Chapter Nine.

There are so many possible arrangements and spacings of successive ramps of freeways, both with and without auxiliary lanes between, that it is not feasible to define all levels of service for each specific combination in this manual. Instead, the general discussion of level of service that follows describes simple junctions where a one-lane ramp connects with a freeway without change in number of freeway lanes. Later in this chapter computational procedures are given for the several levels for the more common of the combinations found on 4-, 6-, and 8-lane freeways. In adapting these procedures to combinations not included here, care should be taken to examine not only the specific junctions involved in the combination but also the entire area of influence upstream and downstream.

On the freeway at the merge or diverge point, level of service A represents unrestricted operation. Entering and exiting traffic has no appreciable effect on through roadway flow, which, on the average, continues at its desired speeds. Entering traffic merges smoothly, with little trouble in finding gaps for merging. At this level under ideal conditions, the total merge (lane 1 plus ramp) does not exceed 1,000 vph. In the case of 4-lane freeways, lanes 1 and 2 combined do not exceed total volumes of 1,400 vph, at about 60 mph.

At level of service B, freeway drivers become conscious of, and adjust for, slight conflicts at entrance ramps, but exit ramps still present no particular problems. Entering traffic must adjust speeds somewhat to fit into freeway lane 1 gaps. Under ideal conditions the total merge (lane 1 plus ramp) does not exceed 1,200 vph. For a 4-lane freeway, lanes 1 and 2 combined are carrying not more than 2,000 vph at speeds of about 55 mph. Operation at exit ramps has little or no effect on the freeway stream, with a diverge service volume of 1,300 vph as an upper limit. Exceptions can occur at poorly designed locations, with an increase in accidents being a probable consequence.

Level of service C represents the limit of assured free flow. It introduces the requirement for consideration of peaking within the peak hour, through use of a peak-hour factor. All drivers are well aware that they are operating in a traffic interchange area, and are prepared to adjust as necessary. They may feel unduly restricted in rural situations, but operation in urban areas is still reasonably acceptable. The maximum total merge (lane 1 plus ramp) varies from 1,300 to 1,550 vph under ideal conditions, depending on the peak-hour factor used, with peak 5-min flow rates equivalent to 1,700 vph. On four-lane freeways lanes 1 and 2 combined are carrying a maximum, depending on peak-hour factor, of 2,300 to 2,750 vph at speeds of about 50 mph. The diverge (lane 1 through vehicles plus prospective exit-ramp vehicles) reaches an upper limit, depending on the peak-hour factor, of 1,400 to 1,650 vph at this level, with peak 5-min flow rates equivalent to 1,800 vph. With good geometrics this volume should be handled with little conflict.

Closely spaced "braided" exit and entrance ramps on freeway near central business district; merging and diverging predominate, there being little weaving demand between left-hand on-ramps and nearby right-hand off-ramps.

In many cases, in levels of service A, B, and C as described in the foregoing, the acceptable merge for that level does not add sufficient traffic to the freeway to change the level of service, or type of operation, radically. At these freeway volume levels ramp volumes will often be correspondingly low. Nevertheless, this is not always the case. Therefore, unless freeway volumes are very low, there are quite distinct limits to the amount of ramp traffic that can be absorbed without exceeding the maximum merge service volumes for the selected level of service. However, at these levels it must be remembered that the merge level of service may be applicable only in the near vicinity of the ramp or ramps involved, rather than along any appreciable length of the freeway upstream or downstream of this location. The remainder is governed only by the total volume and overall operating speeds on the freeway. Of course, closely-spaced ramps can produce a near-continuous influence. Although it is desirable to provide, by design, a uniform level of service over all sections of the freeway, this may not prove feasible for economic reasons. For instance, it may be decided that level of service C can be tolerated at ramp terminals for a distance of 1,000 or 2,000 ft in the middle of a long stretch of highway with level of service A or B.

It should be recognized that with the freeway operating at level of service A, B, or C

there will be an assigned ramp capacity compatible with the level of service chosen for balanced operation. If the ramp volume demand proves larger than that forecast, the excess ramp vehicles will be absorbed into the freeway stream regardless, usually without queuing on the ramp. The difference will be that the level of service of the freeway will drop because of the overloading of the merging and diverging sections, and possibly the through lanes as well, if a predesignated service volume "across all freeway lanes" is exceeded.

Levels of service D, E, and F present a different situation in that demand has become so high that the basic physical ability of the junction to handle increases in demand, or even short-term fluctuations, must be taken into account.

Level of service D represents a condition which is approaching instability and incipient congestion. As this condition is approached, a shift in lane distribution from that found at better levels takes place upstream of the ramp (see p. 234). Ramp volumes considerably higher than those allowable under procedures for level C can be obtained, provided total volume on the freeway does not exceed level D volumes. This being the case, it is the level at which many highway administrators in major cities must work in getting the most out of their existing freeway networks, even though it is above desirable design levels. It is also at or close to the level which is established in certain electronic traffic surveillance procedures as that at which corrective actions should be instituted to prevent "breakdown." Driving conditions are such that reciprocal adjustments in speed and lane occupancy are made by drivers on the freeway as well as on the ramp, but freeway traffic continues to move at average speeds of about 40 mph. Queuing occurs occasionally on the ramp when the ramp volume is relatively heavy. A peak 5-min merge rate equivalent to up to 1,800 vph can be handled, depending on the size of the city, for satisfactory accommodation of short-term fluctuations. A diverge peak rate of 1,900 vph can be accommodated satisfactorily if geometrics are reasonably good. The hourly volumes which will result in these rates, for various peaking factors,

range from 1,400 to 1,650 vph for merging and 1,500 to 1,750 vph for diverging.

Capacity represents, for merging and diverging just as for all other situations, the maximum volume that has reasonable possibility of occurring over a full hour. At a number of ramp terminals in the larger cities, actual hourly merges and diverges of 2,000 to 2,100 passenger cars are regularly accomplished, usually in the 20-30 mph speed range. These sometimes include 5-min rates up to an equivalent 2,300 vph. Therefore, the capacity of both a merging section and a diverging section has been established as 2,000 vph. At this volume operation is at the limit of level of service E. This is not a desirable type of operation, being too unstable and subject to "breakdown," and having intermittent queuing at on-ramps. To obtain capacity merging, the upstream volume must be less than the upper volume limit of level E. That is, unbalanced operation along the freeway is inevitable if substantial merging is introduced at level E. If the approach is already at capacity, input of additional vehicles via on-ramps will inevitably bring on a breakdown in traffic operation.

Level F represents forced flow, which develops following breakdown of merging. Practically all lane 1 and on-ramp traffic is "stop-and-go," with adverse effects moving to other freeway lanes as well as drivers attempt to change lanes to avoid the merging section. Traffic characteristics vary widely, as do merge and diverge volumes, which may be any value below 2,000 vph. At exit ramps, low speed and stop-and-go movement prevent effective accommodation of the demand volume, with considerable delay to drivers likely.

At many locations where this situation has existed for some time, drivers tend to adopt an "alternate feed" (one from the ramp, one from the freeway, and so on). This alternate feed, which usually develops voluntarily on the part of motorists, is an interesting example of motorists familiar with a difficult situation responding courteously in a group effort to keep traffic moving in both arriving queues. In situations such as this, found on crowded freeways, the ramp and lane 1 volumes which can be carried are limited to

approximately 900 vph each. Whether such operation is desirable or not depends on the relative demands on the two approaches. If they are relatively balanced, it may be quite efficient. On the other hand, if they are un-balanced undesirable advantage is given to the lighter flow at the expense of the heavier.

This alternate feed is, in effect, a rudi-mentary example of "metering" of traffic, a procedure gradually coming into greater use. In this simple case, driver courtesy consti-tutes the metering device, producing the fixed regular discharge pattern, regardless of relative traffic demands. In the typical so-phisticated electronic installation, on the other hand, the relative demands on the two or more approaches are continuously moni-tored and interpreted. Vehicles entering the freeway flow are pre-scheduled, by means of controls along the entrance ramp, to reach the merge "at speed" simultaneously with the arrival of a gap in the main line lane 1 flow, thus making optimum use of the merge area with due consideration to the relative vol-umes involved. Insufficient data are yet available to permit presentation of service volumes and capacities associated with such controlled merges.

The basic descriptions of levels of service just presented assume that no additional width beyond the normal through roadway width is provided at any point through the merge or diverge areas. In practice, pro-vision of an auxiliary lane outside the nor-mal width can greatly improve operations where an off-ramp closely follows an on-ramp. In fact, this element is a key to main-tenance of a balanced level of service along the freeway. The auxiliary lane provides the pavement width and maneuvering space necessary to accommodate the temporarily increased volume effectively, without lower-ing the level of service due to increased vol-umes in the through lanes.

It is desirable that such auxiliary lanes be surfaced in a contrasting pavement type, giving clear indication of their specialized purpose, to avoid their utilization by through vehicles.

Table 8.1 summarizes the foregoing fun-damental level of service criteria for simple merge and diverge situations. The table is presented for informational purposes only;

This interchange incorporates a variety of ramp types.

it should not be used directly for capacity and service volume computations without reference to the procedures that are de-scribed later in this chapter. The freeway volume criteria in the table are from Chapter Nine.

When the values in this table are used in conjunction with the procedures for levels A through C to determine on-ramp capabili-ties, there will be cases where the actual de-mand volume on the ramp is found to ex-ceed that allowable at the given merge service volume level. In such cases, one apparent solution is addition of a freeway lane at this point. Another solution is to provide more on-ramps, thus dividing up the load. However, this solution may result in ramps so close together that other unde-sirable operating characteristics are intro-duced. Where this occurs, alternate solu-

TABLE 8.1—SERVICE VOLUMES[a] AND CAPACITY IN VICINITY OF RAMP TERMINALS
(VPH OF MIXED TRAFFIC IN ONE DIRECTION, ASSUMING LEVEL TERRAIN AND NOT OVER 5% TRUCKS)

LEVEL OF SERVICE	FREEWAY VOLUME, ONE DIRECTION[b] (vph)			CHECKPOINT VOLUME (vph)		WEAVING VOLUME[e] (vph)
	4-LANE	6-LANE	8-LANE	MERGE[c]	DIVERGE[d]	
A	1400	2400	3400	1000	1100	800
B	2000	3500	5000	1200	1300	1000
PEAK-HOUR FACTOR[f]	0.77 0.83 0.91 1.00[g]	0.77 0.83 0.91 1.00[g]	0.77 0.83 0.91 1.00[g]	0.77 0.83 0.91 1.00[g]	0.77 0.83 0.91 1.00[g]	0.77 0.83 0.91 1.00[g]
C	2300 2500 2750 3000	3700 4000 4350 4800	5100 5500 6000 6600	1300 1400 1550 1700	1400 1500 1650 1800	1100 1200 1350 1450
D	2800 3000 3300 3600	4150 4500 4900 5400	5600 6000 6600 7200	1400 1500 1650 1800	1500 1600 1750 1900	1400 1500 1650 1800
E[h]	≲4000	≲6000	≲8000	≲2000	≲2000	≲2000
F	←———————————————————— Widely variable ————————————————————→					

[a] Upper limit volume for each level of service.
[b] To be used in making "across all through freeway lanes" service volume checks between ramp-freeway termini and/or interchanges.
[c] Represents the merge taking place, which is determined by the computed lane 1 volume plus the on-ramp volume
[d] Represents the volume in lane 1 immediately upstream from an exit ramp; includes both through vehicles and prospective exit-ramp vehicles.
[e] For weaving between on-ramps and off-ramps per 500 ft of roadway segment.
[f] For freeways, the ratio of the whole-hour volume to the highest hourly rate of flow occurring during a 5-min interval within the peak hour.
[g] A peak-hour factor of 1.00 is rarely attained; the values given should be considered as maximum average flow rates likely to be obtained during the peak 5-min interval within the peak hour.
[h] Capacity.

tions must be considered, keeping in mind that the specified level of service will be applicable only in the near vicinity of the ramp or ramps involved. This being the case, the procedures for level D may be adopted occasionally with reasonable assurance that the level of service upstream or downstream will not be reduced significantly unless the service volume on the main line itself is exceeded. However, drivers will feel somewhat inconvenienced through the ramp junction section.

As previously mentioned, provision of an auxiliary lane may be highly valuable, where an on-ramp precedes an off-ramp, particularly when the ramps are relatively close. The volume between the on-ramp and the off-ramp is always greater than volumes on the adjacent sections, and an auxiliary lane is an inexpensive way to reduce the likelihood that this section will become a bottleneck. The auxiliary lane removes sufficient off-traffic from the freeway, while still carrying much of the on-ramp traffic, to permit the freeway to maintain its intended level of service through the interchange area. Under these conditions the increased width offers considerably increased flexibility for weaving maneuvers.

COMPUTATION PROCEDURES FOR RAMP JUNCTIONS

Procedures for determining acceptable volumes at ramp junctions are based on the premise that if the demand volume does not exceed the service volume at certain critical points, such as those shown in Figure 8.1, which represents a typical junction, good operating conditions will result on both the freeway and ramp and no further detailed analysis is required. These critical point checks include "across all freeway lane" checks and weaving checks, in addition to checks at specific points in specific lanes. Identification of the specific points to be considered is discussed later in the procedures section of this chapter.

If the demand volume somewhat exceeds the service volume for the desired level of service, but remains below capacity, the ramp will still operate, but at a poorer level of service. At the heavier volumes indications of a bottleneck condition may develop. If the demand volume exceeds the capacity
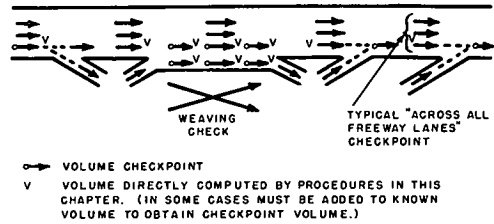
Figure 8.1. Critical points for volume determination in ramp junction analysis.

the bottleneck will be critical and queuing and speed reduction can be expected either on the ramp or on the through freeway lanes, or both.

Determination of ramp junction service volumes involves one of two basic procedures, depending on the particular service level involved. For typical design purposes, or for any situation where assured free flow at level C or better is required, the calculation procedures of the first following section are suggested. For levels D and E, such as in analysis of existing situations (e.g., determining location and cause of bottlenecks) and the checking of designs to be sure that ramp junctions will not cause queuing on the freeway, the calculation procedures of the second following section may be used. In most typical cases making use of these procedures, volumes can be considered as mixed traffic.

In the procedures for levels A through C, equations (or nomographs representing these equations graphically) are employed. As a group, these give consideration to (a) the volume of the ramp under study, (b) the upstream freeway volume, and (c) the distance to and the volume on nearby adjacent ramps, both upstream and downstream. Not all of these variables are used in each equation or nomograph, however. In the case of entrance ramps these factors establish the expected merge, which consists of the sum of the computed lane 1 volume on the freeway at the on-ramp nose and the assigned ramp volume. In the case of simple exit ramps the same variables apply; they are used to determine a lane 1 volume immediately upstream from the exit ramp. This computed volume

includes lane 1 vehicles continuing on the freeway past the exit ramp, as well as prospective exit-ramp vehicles.

Although a wide variety of combinations of geometrics is covered by the equations and nomographs, not all possible designs are included, for lack of adequate study data. Several such omissions are discussed, and rationalized computation procedures are suggested.

In the procedures for levels D and E, use is made of standard charts that establish lane distribution criteria at various points along the merge or diverge area, to establish volumes in lane 1 which can be compared with criteria for level of service D and capacity. This procedure can be used for an isolated single-lane ramp or any combination of single-lane ramps with or without auxiliary lanes.

In both procedures, volumes are considered as representing mixed traffic containing not over 5 percent trucks, on relatively level terrain with grades not over 3 percent. This represents the average condition base upon which the methods were developed. Where substantially greater truck volumes or steeper grades are involved, the procedures involving the equations and nomographs should still be carried out without change; only the final result should be adjusted for trucks and grades, as a final additional step, as discussed later in this chapter under "Related Computational Devices."

Calculation of Service Volumes, Levels A Through C

GENERAL PROCEDURES

In the usual case the engineer who is evaluating either an existing interchange design or a proposed new design is given the traffic demand for each of the movements involved. His need is to determine whether or not the design will operate satisfactorily at the level of service he is considering.

In most cases a design will be considered satisfactory if it permits operation at level C, representing relatively free flow, or better.

Conditions which can be expected if the demand volume does not exceed level C for the merge or diverge include a good operating condition at the freeway terminal, no significant queuing upstream of the entrance

ramp terminal, no significant speed reduction on the through freeway lanes as a result of the ramp traffic being added, and no significant effect on through traffic by exiting vehicles at off-ramps. This is not to say that queuing and speed reduction would never occur, as there is always a possibility, at an on-ramp, of a queue of traffic arriving on the ramp at the same time that a queue occurs on the freeway. It is practically certain that this will happen at any ramp occasionally, at almost any volume level, although not as frequently at the lower volumes. This type of operation at ramps must be expected and should not be considered an unsatisfactory freeway operation; it cannot be "designed out" by assuming lower design volumes. Failure occurs when the queue does not dissipate quickly but exists for several minutes.

This section, then, discusses procedures for determination of the lane 1 volume and the resultant merge and diverge service volumes that can be handled at level C or better. They are equally suitable for levels A, B, and C. If preliminary analysis indicates that operation at or near capacity is involved, and if local conditions permit no other alternative but such operation, then operating conditions may be evaluated as outlined for levels D and E, later in this chapter, instead of by this method.

The method involves calculation of various lane 1 volumes by means of equations which were developed by multiple regression techniques for the purpose of estimating the probable traffic volume in lane 1 of a freeway at selected checkpoints (1, 2). For convenience of use these equations also are presented as nomographs.

The steps are as follows:

1. Establish the geometrics of the location under study, including number of freeway lanes and location and type of adjacent ramps upstream and downstream from the junction under study. In the case of new designs, this may involve several trial designs. (If necessary, compute the number of freeway lanes by means of the procedures given in Chapter Nine.)

2. Establish the demand volumes for all traffic movements involved.

3. Select the appropriate equation (or

nomograph) for the geometrics involved, and compute the expected volume in lane 1 at the appropriate checkpoint or points (or other dependent variable in certain designs).

4. Analyze the criteria assembled in the previous steps, as follows (adjusting for trucks and grades as a final step, if necessary, by the procedure given under "Related Computational Devices" later in this chapter):

(a) At a merge point—The anticipated on-ramp volume is added to the computed lane 1 volume at the nose to give an expected merge, which is compared with the merge checkpoint maximum allowable service volume given in Table 8.1.

(b) At a diverge point—The computed lane 1 volume immediately upstream from an exit ramp, which includes through vehicles remaining in this lane as well as vehicles about to exit, is compared to the diverge checkpoint service volume from

Table 8.1. Usually, the exit ramp volume is an assigned volume, in which case the difference between the computed lane 1 volume and the assigned exit ramp volume is the number of through vehicles. However, there will also be cases where the designer wishes to know how many vehicles can exit at a ramp given a certain freeway volume approaching the ramp and a lane 1 diverge service volume which must not be exceeded. In this type of problem, the dependent variable (the lane 1 volume) is already given, so the allowable ramp volume for the chosen level of service can be computed directly.

(c) At a location with an auxiliary lane—Where an auxiliary lane is added between an entrance ramp and an exit ramp, lane 1 and auxiliary lane volumes are calculated at selected points between the ramps. These volumes are checked against the merge service volume or the diverge service volume de-

Left-side direct-connection ramps as used together with right-side outer connections in this interchange between two freeways.

pending on the location of the checkpoints. These procedures are discussed in detail later under "Auxiliary Lane Use."

(d) For weaving between closely spaced ramps—Weaving service volume criteria are presented in Table 8.1. The assigned ramp volumes are added and checked against the table value. If necessary, calculations can be performed to determine the weaving taking place on any 500-ft segment of the weaving section.

(e) For 2-lane ramps and major forks— The various merge and diverge volumes which can be checked are compared with the appropriate standard merge and diverge service volumes in Table 8.1.

(f) For "across all through freeway lanes" volumes—The total volume of traffic on the freeway, excluding auxiliary lane volumes, is checked against the freeway service volume given in Table 8.1. In this procedure any auxiliary lane is not counted as a freeway lane.

5. Evaluate and interpret the results of the analyses in Step 4, as follows:

Service volumes for the particular level of service selected should not be exceeded at any point if full harmony of design is to be maintained. If they are not exceeded the design is considered satisfactory for traffic operation at the selected level of service. If service volumes are exceeded at one or more points:

(a) In the case of a new design, redesign if at all possible, or accept a poorer level of service. In considering the latter alternative, remember that, although an occasional junction operating somewhat below the desired level may be acceptable, the more often this alternative is used the poorer will be the overall level of service on the highway.

(b) In the case of an existing facility, consider reconstruction or accept a restricted level of service. At important locations, consider special metering or other control measures capable of developing highly efficient junction operations.

The usual goal of redesign or reconstruction steps to be taken if service volumes are exceeded is to reduce checkpoint volumes to within the selected level of service ranges. Examples of possible methods would include:

(a) Addition of an auxiliary lane.

(b) Increase in the distance between ramps or interchanges.

(c) Separation of high-volume ramps into two ramps.

(d) Use of a collector-distributor road to separate weaving from mainline traffic.

(e) Rearrangement of ramp sequence (possibly involving braiding of ramps).

(f) Addition of a freeway lane or lanes.

VARIABLES CONSIDERED

The equations to be presented involve, as a group, a substantial number of factors and variables, although not all are used in any one equation. These are here defined and described:

1. Lane Designations

Lane 1.—As defined throughout this manual, the right-hand through lane of the freeway.

Ramp Lane A.—The ramp lane closest to the freeway, in the case of a two-lane ramp. In the case of a major fork, it would be the lane of the merging or diverging roadway closest to the adjacent roadway.

Ramp Lane B.—The ramp lane farthest from the freeway, in the case of a two-lane ramp. In the case of a major fork, it would be the lane of the merging or diverging roadway farthest from the adjacent roadway.

2. Dependent Variables

The equations for 4-, 6-, and 8-lane freeways involve three separate dependent variables, all measures of volume. These give volumes in vehicles per hour (vph) of mixed traffic, containing up to 5 percent trucks. However, if properly interpreted they can be considered as hourly rates expanded from short-period counts. The equations yield estimates of lane 1 volumes which are subject to standard errors of estimate. The variables are as follows:

$V_1 =$ (1) for an on-ramp equation, the lane 1 volume at the on-ramp nose just before the merge takes place.

(2) for an off-ramp equation, the lane 1 volume upstream from the off-ramp nose, immediately before divergence takes place.

(3) for a 2-lane off-ramp or a major

fork, the lane 1 volume immediately downstream from the bifurcation.

$V_{1+A} =$ (1) For a 2-lane on-ramp, the initial merge volume of lane 1 vehicles and ramp lane A vehicles (*i.e.*, the left ramp lane or lane closest to the freeway lanes).

(2) for a 2-lane off-ramp, the volume in lane 1 of the combined flow before the divergence takes place which splits the volume into freeway lane 1 and ramp lane A.

$V_c =$ for a major fork on a 6-lane freeway, the volume in the center lane before it splits into lane 1 and lane A of the two fork legs, respectively.

3. Independent Variables

The seven independent variables used, as appropriate, in the several equations, follow. Again, volumes are in vehicles per hour of mixed traffic, with up to 5 percent trucks.

$V_f =$ for an on-ramp equation, the freeway volume, total for all freeway lanes, in one direction immediately upstream of the nose of the on-ramp before the merge takes place.

$V_t =$ for an off-ramp equation, the total freeway volume, including prospective off-ramp vehicles, upstream of (approaching) the off-ramp.

$V_r =$ (1) for an on-ramp equation, the volume entering via the ramp being considered in the prospective merge.

(2) for an off-ramp equation, the volume exiting at the off-ramp under consideration.

(3) for a major fork, the volume using the right-hand roadway.

$D_u =$ distance, in feet, measured as in Figure 7.5, from the ramp under consideration to an adjacent upstream on-ramp or off-ramp.

$V_u =$ volume on an adjacent upstream on-ramp or off-ramp.

$D_d =$ distance, in feet, measured as in Figure 7.5, from the ramp under consideration to an adjacent downstream on-ramp or off-ramp. Where an auxiliary lane is added between ramp noses, this distance is identical to the length of the auxiliary lane.

$V_d =$ volume on an adjacent downstream on-ramp or off-ramp.

COMPUTATIONAL EQUATIONS AND NOMO-GRAPHS, LEVELS A THROUGH C

Figures 8.2 through 8.19 present 18 equations and 18 equivalent nomographs for use in determining lane 1 volumes on 4-, 6-, and 8-lane freeways, primarily at levels of service A through C, given a variety of ramp-freeway junction geometric layouts for which sufficient data were available for analysis. To the extent found significant in any particular layout, these equations take into account the distances to and the volumes on adjacent upstream and downstream ramps, as well as the freeway and ramp volumes at the terminal for which the computations are being made. Special conditions and limitations on use are shown in some cases. Detailed statistical data regarding the multiple regression analyses employed to develop these equations are given in Appendix C.

Where the method shows a given design to be unsatisfactory, the remedies involve largely exploratory analyses. No direct indication is given of what steps to take, but other designs must be assumed and tested. Clues are provided, however, which the designer can interpret as related to basic freeway operating characteristics. He knows, for instance, that once he has made one computation which resulted in an unsatisfactory merge, he must revise the design (such as by providing an auxiliary lane or by moving a ramp), reduce the given volume of ramp traffic (by providing other paths for part of the demand), or reduce lane 1 volumes. Inasmuch as certain of the factors covered by the equations will be constant for the particular site under study, he can, by inspection, quite rapidly ascertain the alternatives open to him. For convenience in these calculations, the series of nomographs representing each of the equations has been included. Instructions for the use of each are given directly on the charts. Their use will frequently expedite rapid evaluation of the several potential alternatives in a particular case.

For most purposes, as already indicated, the volumes as used and as computed in these procedures can be taken as mixed traffic containing a few trucks (at least up to 5 percent) without serious error. However, where grades are significant or truck volumes are substantial, allowance should be made for them through adjustment of the final result by application of the truck equivalency factors presented in Chapter Nine. (Adjustments should not be made in the intermediate computations; the mixed traffic volumes should be used directly.)

Following the series of figures is a group of related computational devices to which reference must occasionally be made in using certain of the figures. Also included is discussion of other geometric combinations for which insufficient data were available for development of specific equations. To the extent possible, approximate procedures for handling such cases are described.

Summary Table 8.2 is provided as a convenient index to the geometric combinations covered in the chapter, by means of either figure or discussion.

RELATED COMPUTATIONAL DEVICES

Auxiliary Lane Use.—Figures 8.6, 8.7, 8.11, 8.12, and 8.16 are used for on-ramp locations having an auxiliary lane extending to the adjacent downstream off-ramp. The presence of an auxiliary lane changes the computational procedures somewhat from those used in conventional merging and diverging situations. At auxiliary lane locations, the extended opportunity to weave or change lanes between lane 1 and the auxiliary lane makes necessary a computation of volume in each of these lanes at selected points between the ramp noses. Also, checks of weaving volume per 500 ft of roadway should be made.

The computed lane 1 and auxiliary lane volumes should be checked separately against the service volume. If the checkpoint is at one-half the distance between the ramps, or closer to the on-ramp, the merge service volume should be used. If closer to the off-ramp, the diverge service volume should be used.

In making an "across all freeway lanes" volume check, the auxiliary lane should not be counted as a lane and the volume on the auxiliary lane should not be included in the total volume considered.

Figure 8.20 is an additional device used in conjunction with the equations in the analysis of auxiliary lane locations. Its use permits examination of the status of "on" and "off" transitional movements at any point along the auxiliary lane. It is intended for application to auxiliary lanes 1,400 ft or less in length, as covered by Figures 8.6, 8.7, 8.11, 8.12, and 8.16. (If the auxiliary lane is longer than 1,400 ft, the distributions shown in Case II of Figure 8.23 can be applied.) The computational procedure used is as follows:

(a) Determine lane 1 volume at the on-ramp nose by the use of the appropriate basic nomograph (Fig. 8.6, 8.7, 8.11, 8.12, or 8.16). This lane 1 volume will consist of lane 1 through vehicles and vehicles intending to exit at the next off-ramp downstream. For reasons of simplicity in figuring the lane 1 through volume, 100 percent of the "intending to exit" off-ramp vehicles are considered to be in lane 1 at the on-ramp nose. In practice, this is more likely to be approximately 95 percent, as there are always a few "late decision," "blocked off," or "sleeping" prospective off-ramp drivers, who are still in lane 2 at the on-ramp nose.

(b) Subtract the off-ramp volume from the computed lane 1 volume to get the lane 1 through volume.

(c) Make several checks of lane 1 and auxiliary lane volumes at points between the ramps. The volumes consist of the following:

Lane 1 volume = Lane 1 through + On-ramp vehicles out of auxiliary lane (Fig. 8.20, upper curve) + Off-ramp vehicles still in lane 1 (Fig. 8.20, interpreted from lower curve).

Auxiliary lane volume = On-ramp vehicles still in auxiliary lane (Fig. 8.20, interpreted from upper curve) + Off-ramp vehicles which have moved onto the auxiliary lane (Fig. 8.20, lower curve).

Because lane 1 carries through vehicles as well as ramp vehicles, it seems obvious that it will usually be the critical lane in terms of

GEOMETRIC ARRANGEMENT[b,c]	4-LANE FREEWAY (2 LANES EACH DIRECTION)		6-LANE FREEWAY (3 LANES EACH DIRECTION)		8-LANE FREEWAY (4 LANES EACH DIRECTION)	
	ON-RAMP	OFF-RAMP	ON-RAMP	OFF-RAMP	ON-RAMP	OFF-RAMP
ONE-LANE RAMPS	Fig. 8.2, or Fig. 8.8	—	Fig. 8.13, (or Fig. 8.9—see (f), p. 225)	—	Fig. 8.14 or Fig. 8.15	—
	(Fig. 8.2)	—	Fig. 8.9	—	(Fig. 8.14 or Fig. 8.15)	—
	—	Fig. 8.3 or Fig. 8.4	—	Fig. 8.10	—	(Table 8.3 and Fig. 8.24b)
	Fig. 8.4	(Fig. 8.4)	—	—	—	—
	Fig. 8.6	(Fig. 8.6 and Fig. 8.20)	Fig. 8.11	(Fig. 8.11 and Fig. 8.20)	(Fig. 8.16)	(Table 8.3 and Fig. 8.24b)
	Fig. 8.7	(Fig. 8.7 and Fig. 8.20)	Fig. 8.12	(Fig. 8.12 and Fig. 8.20)	Fig. 8.16	(Table 8.3 and Fig. 8.24b)
1ST 2ND	1st—Fig. 8.2 2nd—Fig. 8.8 or Fig. 8.2	—	1st—Fig. 8.9 2nd—Fig. 8.13	—	(Table 8.3 and Fig. 24b)	—
1ST 2ND	—	1st—(Fig. 8.4 and (a), p. 225, Col. 2) 2nd—(Fig. 8.3 or 8.4)	—	1st—(Fig. 8.10 and (b), p. 226, top) 2nd—(Fig. 8.10)	—	(Table 8.3 and Fig. 8.24b)
	(Refs. 9 and 10)					
TWO-LANE RAMPS	Not available	Not available	Fig. 8.17	—	Not available	Not available
	Not available	Not available	—	Fig. 8.18	Not available	Not available
VARYING NUMBER OF LANES (INCLUDING MAJOR JUNCTIONS AND FORKS)						
ONE-LANE RAMPS	(Table 8.1—see (a), p. 226)	—	(Table 8.1—see (a), p. 226)	—	(Table 8.1—see (a), p. 226)	—
	—	(Table 8.1—see (a), p. 226)	—	(Table 8.1—see (a), p. 226)	—	(Table 8.1—see (a), p. 226)
MAJOR JUNCTIONS AND FORKS	(Case 1, p. 226)	—	(Case 1, p. 226)	—	(Case 1, p. 226)	—
	—	Not available	--	Fig. 8.19	—	Not available

[a] Entries in parentheses indicate suggested adaptations of criteria not developed specifically for the geometrics shown, and/or references to discussion.
[b] Specific ramp junctions under consideration are emphasized.
[c] Acceleration or deceleration lanes not shown.
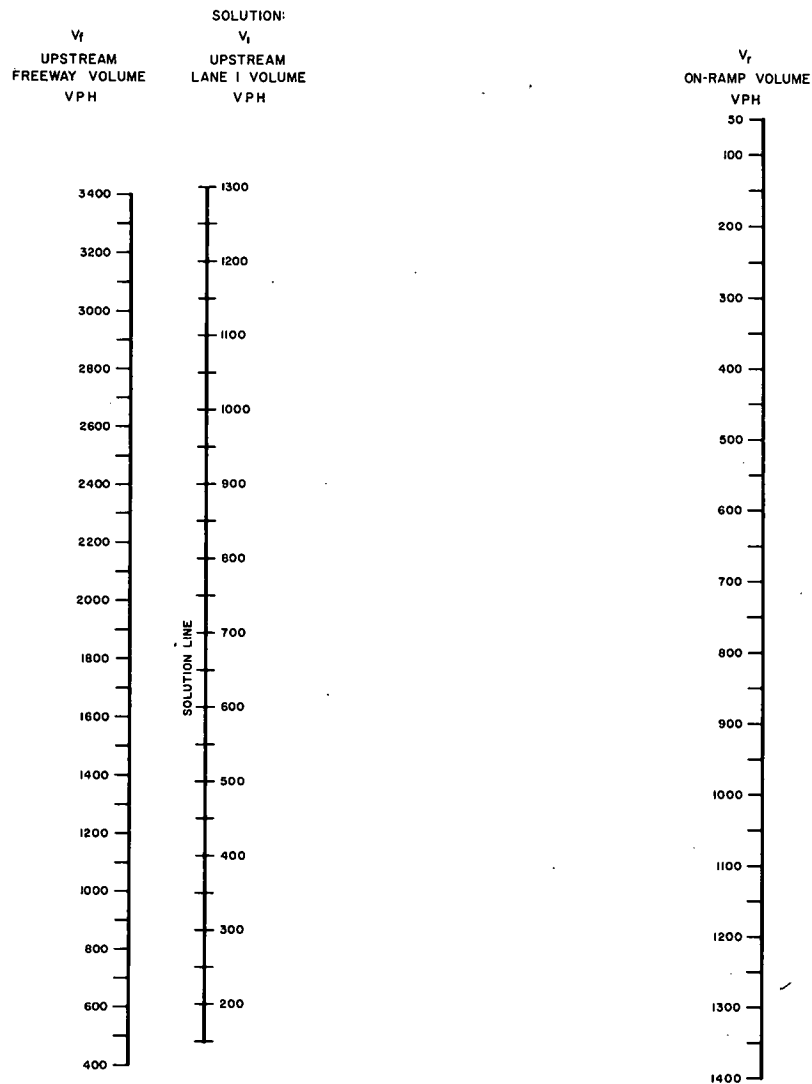
SOLUTION:

V_f
UPSTREAM
FREEWAY VOLUME
VPH

V_1
UPSTREAM
LANE I VOLUME
VPH

V_r
ON-RAMP VOLUME
VPH

SOLUTION LINE

$V_1 = 136 + 0.345\ V_f - 0.115\ V_r$

STANDARD ERROR = 113 VPH
LIMITATIONS:
V_f : 400 – 3400 VPH
V_r : 50 – 1400 VPH
NO UPSTREAM ON-RAMP
WITHIN 2,000 FEET

CONDITIONS FOR USE

To determine the lane 1 volume at an on-ramp nose before merging takes place. The ramp can be of any single-lane type except cloverleaf loop (handled by Figs. 8.5 and 8.6). An acceleration lane may or may not be present.

If there is an adjacent upstream on-ramp within 2,000 ft. use Fig. 8.8.

STEPS IN SOLUTION

Draw a line from V_f value to V_r value intersecting V_1 on Solution Line.

Figure 8.2. Nomograph for determination of lane 1 volume upstream of on-ramp junction, 4-lane freeway (not applicable to cloverleaf inner loop).

V_f
UPSTREAM
FREEWAY VOLUME
VPH

V_1
UPSTREAM
LANE 1 VOLUME
VPH

V_r
OFF-RAMP VOLUME
VPH

4200

4000

3800 — 2300

3600 — 2200

— 2100

3400 — 2000

— 1900

3200 — 1800

3000 — 1700

2800 — 1600

2600 — 1500

2400 — 1400

2200 — 1300

2000 — 1200

1800 — 1100

1600 — 1000

1400 — 900

— 800

1200 — 700

1000 — 600

800 — 500

600 — 400

400 — 300

SOLUTION LINE

1500

1400

1300

1200

1100

1000

900

800

700

600

500

400

300

200

100

0

$V_1 = 165 + 0.345\ V_f + 0.520\ V_r$

STANDARD ERROR = 131 VPH
LIMITATIONS :
V_f : 400 – 4200 VPH
V_r : 50 – 1500 VPH
NO UPSTREAM ON-RAMP
WITHIN 3200 FEET

CONDITIONS FOR USE

To determine the lane 1 volume upstream from an off-ramp just before divergence takes place. The ramp may or may not have a deceleration lane. If there is an adjacent upstream on-ramp within 3,200 ft, Fig. 8.4 should be used instead for greater accuracy.

Note: See p. 222 for refinements possible in the use of this equation.

STEPS IN SOLUTION

Draw a line from V_f value to V_r value intersecting V_1 on Solution Line.

Figure 8.3. Nomograph for determination of lane 1 volume upstream of off-ramp junction, 4-lane freeway (no upstream on-ramp within 3,200 ft of off-ramp).

V_f
UPSTREAM
FREEWAY
VOLUME
VPH

4200
4000
3800
3600
3400
3200
3000
2800
2600
2400
2200
2000
1800
1600
1400
1200
1000
800
700

SOLUTION:
V_t
UPSTREAM
LANE 1 VOLUME
VPH

2250
2200
2100
2000
1900
1800
1700
1600
1500
1400
1300
1200
1100
1000
900
800
700
600
500
400
300
250

TURNING LINE 1

TURNING LINE 2

SOLUTION LINE

V_r
OFF-RAMP VOLUME
VPH

1600
1500
1400
1300
1200
1100
1000
900
800
700
600
500
400
300
200
100
25

$$V_t = 202 + 0.362\ V_f + 0.496\ V_r$$
$$-0.069\ D_u + 0.096\ V_u$$

STANDARD ERROR = 83 VPH
LIMITATIONS :
V_f : 70 – 4200 VPH
V_r : 50 – 1600 VPH
V_u : 50 – 900 VPH
D_u : 700 – 3200'

CONDITIONS FOR USE

Same as for Fig. 8.3, except that there is an upstream, adjacent
on-ramp within 3,200 ft.

STEPS IN SOLUTION

(1) Draw a line from V_f value to V_r value intersecting Turning Line 1.
(2) Draw a line from Step 1 intersection of Turning Line 1 to D_u value,
intersecting Turning Line 2.
(3) Draw a line from Step 2 intersection of Turning Line 2 to V_u value.
The intersection point of this line with the Solution Line is V_1.

D_u
DISTANCE TO UPSTREAM
ON-RAMP
(FEET)

700
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
2800
3000
3200

V_u
VOLUME OF
UPSTREAM
ON-RAMP
VPH

900
800
700
600
500
400
300
200
100
50

Figure 8.4. Nomograph for determination of lane 1 volume upstream of off-ramp junction, 4-lane freeway, with upstream on-ramp within 3,200 ft of off-ramp (no auxiliary lane).
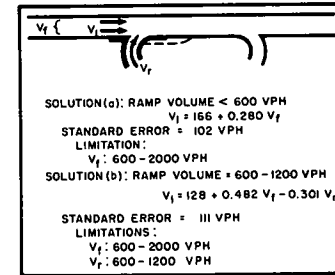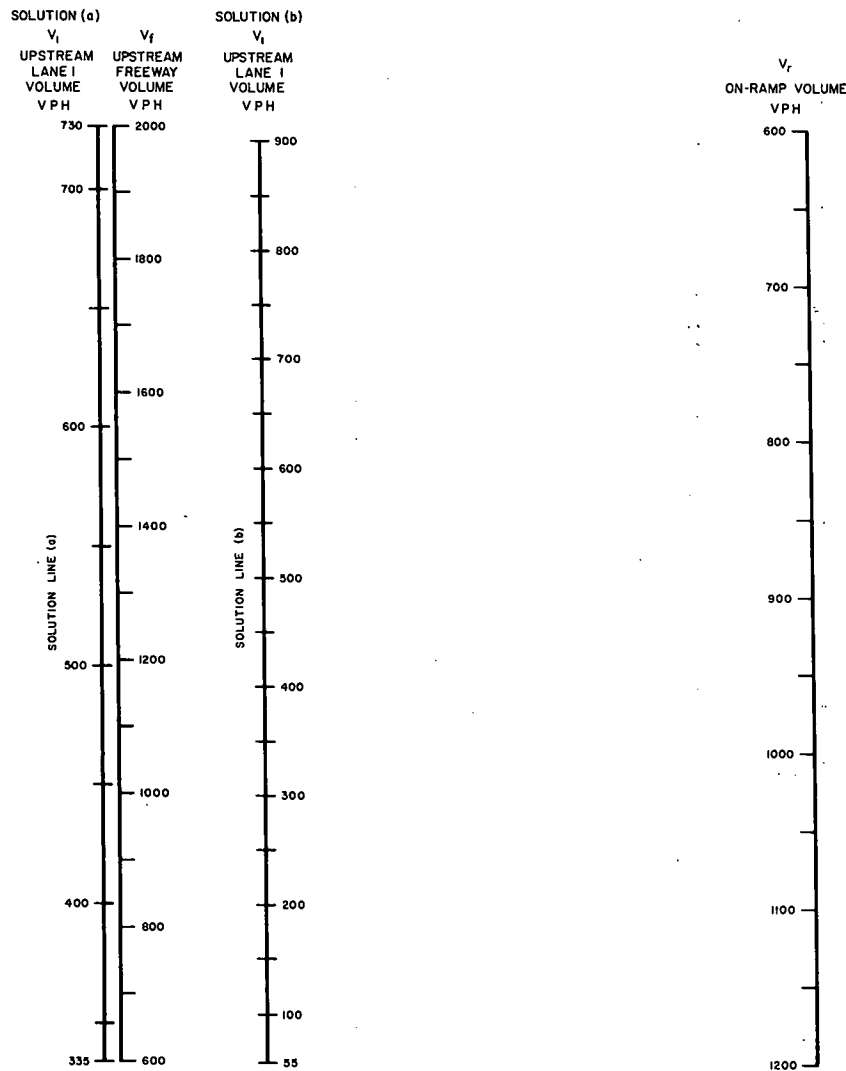
SOLUTION (a)

V_i
UPSTREAM
LANE I
VOLUME
V P H

V_f
UPSTREAM
FREEWAY
VOLUME
V P H

SOLUTION (b)

V_i
UPSTREAM
LANE I
VOLUME
V P H

V_r
ON-RAMP VOLUME
V P H

SOLUTION LINE (a)

SOLUTION LINE (b)

SOLUTION(a): RAMP VOLUME < 600 VPH
$V_i = 166 + 0.280\ V_f$
STANDARD ERROR = 102 VPH
LIMITATION:
V_f: 600 - 2000 VPH
SOLUTION(b): RAMP VOLUME = 600 - 1200 VPH
$V_i = 128 + 0.482\ V_f - 0.301\ V_r$

STANDARD ERROR = 111 VPH
LIMITATIONS:
V_f: 600 - 2000 VPH
V_r: 600 - 1200 VPH

CONDITIONS FOR USE

To determine the lane 1 volume at an inner loop on-ramp nose at a cloverleaf interchange which lacks an auxiliary lane connecting the loop ramps. An acceleration lane may or may not be present. This design, lacking an auxiliary lane, is infrequently found along modern freeways, but a number of such interchanges are still in use on older freeways. Solution (a) is appropriate where the on-ramp volume is less than 600 vph. Solution (b) is used where the on-ramp volume is 600-1,200 vph.

STEPS IN SOLUTION

Solution (a)
For ramp volumes less than 600 vph
V_1, on Solution Line (a), is horizontally to the left of V_f scale.
Solution (b)
For ramp volumes between 600 and 1200 vph
Draw a line from V_f value to V_r value intersecting V_1 on Solution Line (b).

Figure 8.5. Nomograph for determination of lane 1 volume upstream of on-ramp junction, 4-lane freeway, at cloverleaf inner loop (no auxiliary lane).
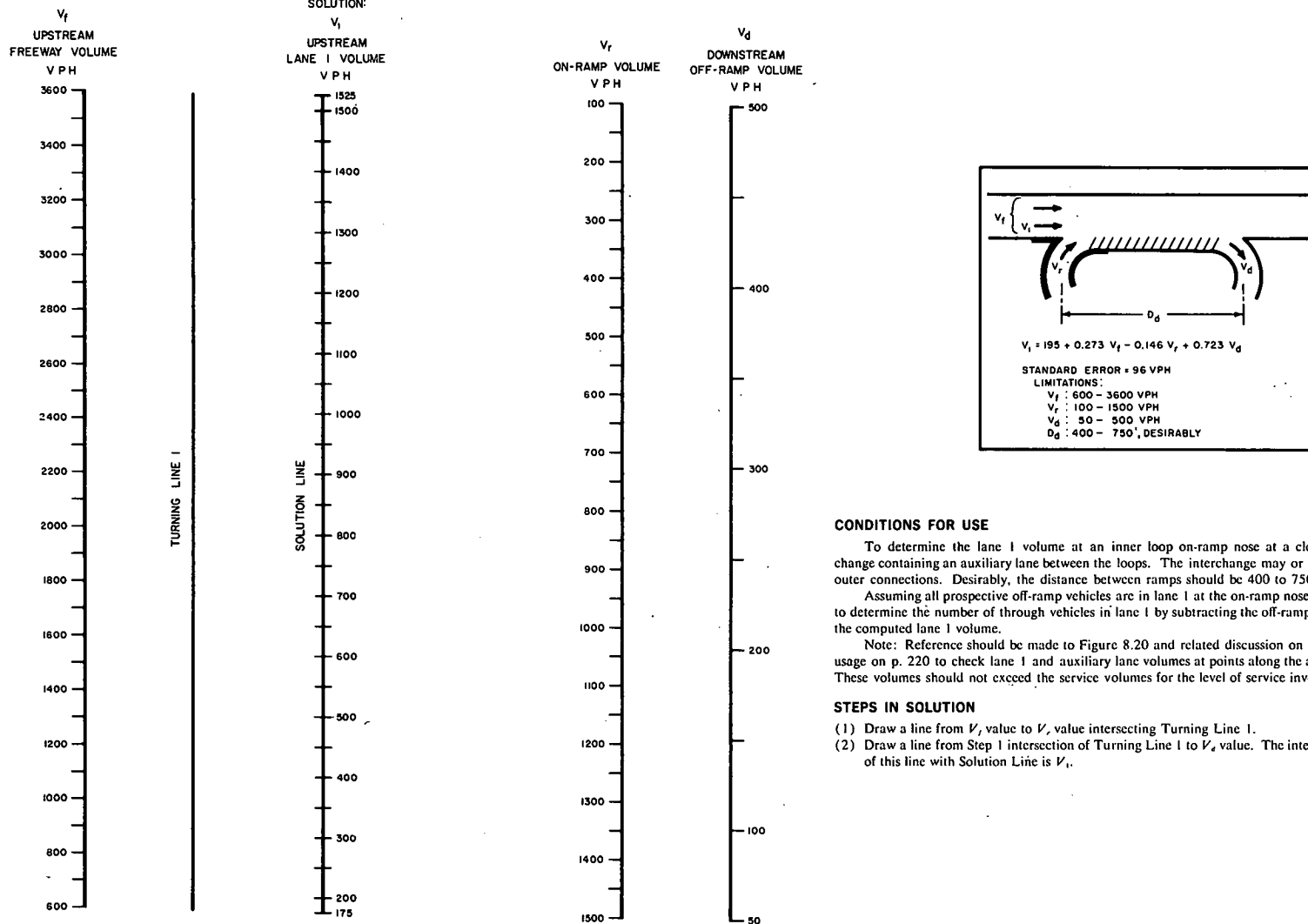
V_f
UPSTREAM
FREEWAY VOLUME
V P H

SOLUTION:
V_1
UPSTREAM
LANE I VOLUME
V P H

V_r
ON-RAMP VOLUME
V P H

V_d
DOWNSTREAM
OFF-RAMP VOLUME
V P H

TURNING LINE I

SOLUTION LINE

$V_1 = 195 + 0.273 \, V_f - 0.146 \, V_r + 0.723 \, V_d$

STANDARD ERROR = 96 VPH
LIMITATIONS:
V_f : 600 – 3600 VPH
V_r : 100 – 1500 VPH
V_d : 50 – 500 VPH
D_d : 400 – 750', DESIRABLY

CONDITIONS FOR USE

To determine the lane 1 volume at an inner loop on-ramp nose at a cloverleaf interchange containing an auxiliary lane between the loops. The interchange may or may not have outer connections. Desirably, the distance between ramps should be 400 to 750 ft.

Assuming all prospective off-ramp vehicles are in lane 1 at the on-ramp nose, it is possible to determine the number of through vehicles in lane 1 by subtracting the off-ramp volume from the computed lane 1 volume.

Note: Reference should be made to Figure 8.20 and related discussion on auxiliary lane usage on p. 220 to check lane 1 and auxiliary lane volumes at points along the auxiliary lane. These volumes should not exceed the service volumes for the level of service involved.

STEPS IN SOLUTION

(1) Draw a line from V_f value to V_r value intersecting Turning Line 1.
(2) Draw a line from Step 1 intersection of Turning Line 1 to V_d value. The intersection point of this line with Solution Line is V_1.

Figure 8.6. Nomograph for determination of lane 1 volume upstream of on-ramp junction, 4-lane freeway, at cloverleaf inner loop with auxiliary lane.

Figure 8.7. Nomograph for determination of lane 1 volume upstream of on-ramp junction, 6-lane freeway, with adjacent off-ramps both upstream and downstream of stream off-ramp.

V_f

V_l

FREEWAY VOLUME
UPSTREAM OF
2 ND ON-RAMP
VPH

LANE I VOLUME
UPSTREAM OF
2 ND ON-RAMP
VPH

V_r

2 ND ON-RAMP
VOLUME
VPH

V_f	V_l	V_r
3600	1460	100
3400	1400	200
3200	1300	300
3000	1200	400
2800	1100	500
2600	1000	600
2400	900	700
2200	800	800
2000	700	900
1800	600	1000
1600	500	1100
1400	400	1200
1200	300	1300
1000	250	1400
800	210	1500

SOLUTION LINE

$$V_l = 123 + 0.376\ V_f - 0.142\ V_r$$

STANDARD ERROR = 76 VPH

LIMITATIONS :
V_f = 800 – 3600 VPH
V_r = 100 – 1500 VPH
V_u = 100 – 1000 VPH
D_u = 400 – 2000'

2 ND ON-RAMP

V_f { V_l

V_u

V_r

D_u

CONDITIONS FOR USE

To determine the lane 1 volume at the on-ramp nose of the second of successive on-ramps when the ramps are within 400 to 2,000 ft of each other (use Fig. 8.2 otherwise), and the upstream on-ramp volume does not exceed 1,000 vph. Not applicable to situations where the upstream on-ramp volume, V_u, is near its maximum of 1,000 vph, and the distance between ramps, D_u, is near its minimum of 400 ft. Thus, the variables V_u and D_u, while not directly incorporated in the equation, nevertheless must be within a specified range for accurate use of the equation.

STEPS IN SOLUTION

Draw a line from V_f value to V_r value intersecting Solution Line at V_l.

Figure 8.8. Nomograph for determination of lane 1 volume upstream of on-ramp junction, 4-lane freeway, at second of successive on-ramps.
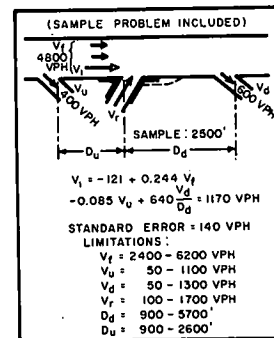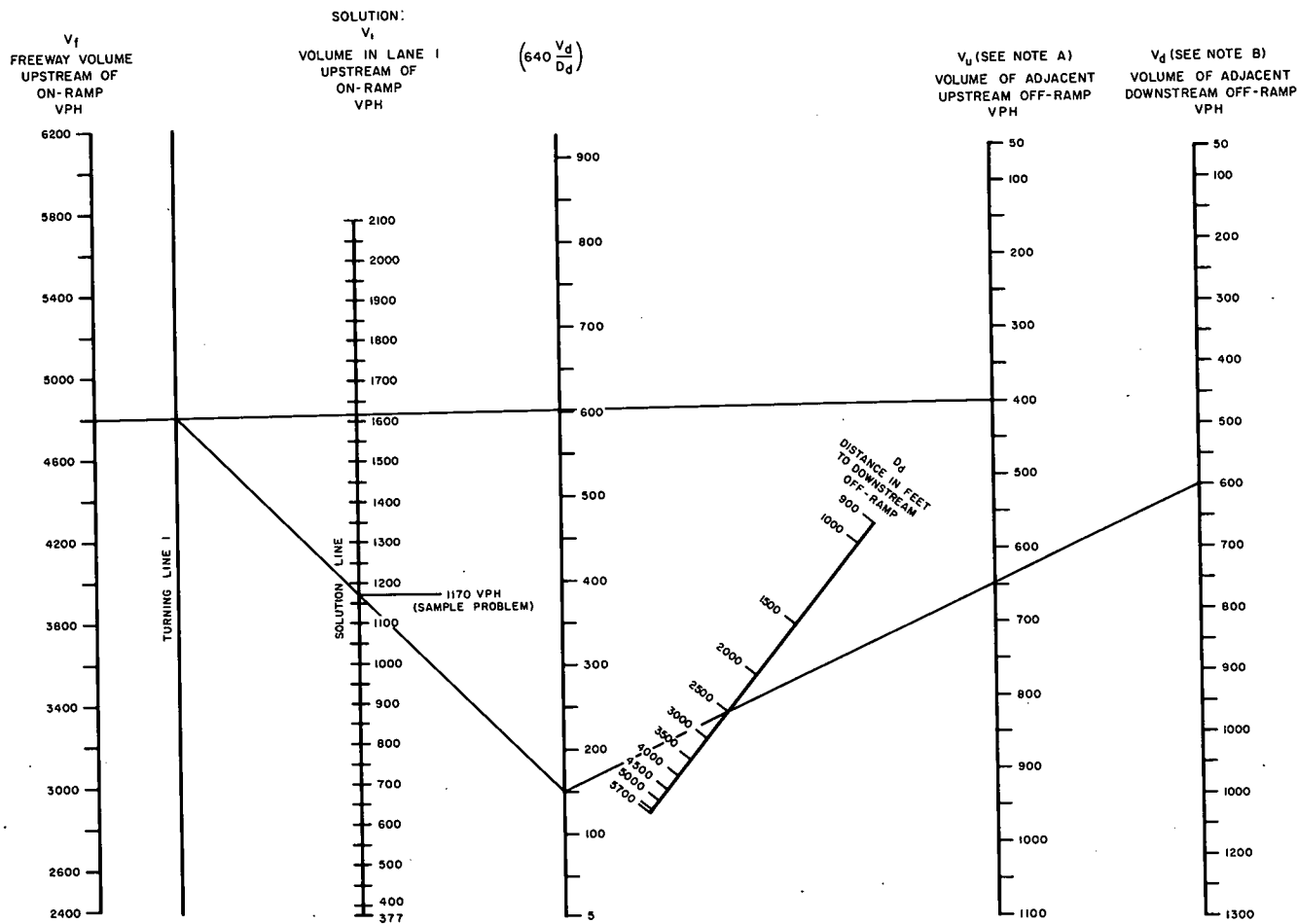
V_f
FREEWAY VOLUME
UPSTREAM OF
ON-RAMP
VPH

SOLUTION:
V_1
VOLUME IN LANE I
UPSTREAM OF
ON-RAMP
VPH

$\left(640\dfrac{V_d}{D_d}\right)$

V_u (SEE NOTE A)
VOLUME OF ADJACENT
UPSTREAM OFF-RAMP
VPH

V_d (SEE NOTE B)
VOLUME OF ADJACENT
DOWNSTREAM OFF-RAMP
VPH

(SAMPLE PROBLEM INCLUDED)

V_f
4800
VPH

V_u
900 VPH

SAMPLE: 2500'

D_u D_d

$V_1 = -121 + 0.244\,V_f$
$-0.085\,V_u + 640\dfrac{V_d}{D_d} = 1170$ VPH

STANDARD ERROR = 140 VPH
LIMITATIONS:
V_f = 2400 - 6200 VPH
V_u = 50 - 1100 VPH
V_d = 50 - 1300 VPH
V_r = 100 - 1700 VPH
D_d = 900 - 5700'
D_u = 900 - 2600'

CONDITIONS FOR USE

To determine the lane 1 volume at an on-ramp nose
before merging takes place. The ramp can be of any
single-lane type, except cloverleaf loop (see Fig. 8.11).
An acceleration lane may or may not be present. While
not included in the equation, the volume on the ramp
under study and the distance to the adjacent upstream
off-ramp should be within the limits given.

Notes: A. If there is no upstream off-ramp within
2,600 ft, use 50 on the V_u scale when drawing Step 1 line.
B. If there is no downstream off-ramp within 5,700 ft
and V_f value does not exceed 5,000 vph, skip Step 2 and
use 5 on the $\left(640\dfrac{V_d}{D_d}\right)$ scale from which to draw Step 3
line.

STEPS IN SOLUTION

(1) Draw line from V_f value to V_u value, intersecting
Turning Line I.
(2) Draw line from V_d value through D_d value to inter-
sect $\left(640\dfrac{V_d}{D_d}\right)$ line.
(3) Draw line from this value on $\left(640\dfrac{V_d}{D_d}\right)$ line to
Step 1 intersection of Turning Line 1. The inter-
section on the Solution Line is V_1.

TURNING LINE 1

SOLUTION LINE

DISTANCE IN FEET
TO DOWNSTREAM
OFF-RAMP D_d

1170 VPH
(SAMPLE PROBLEM)

Figure 8.9. Nomograph for determination of lane 1 volume upstream of on-ramp junction, 6-lane freeway, with adjacent off-ramps both upstream and downstream of on-ramp (no auxiliary lanes).
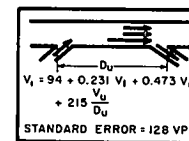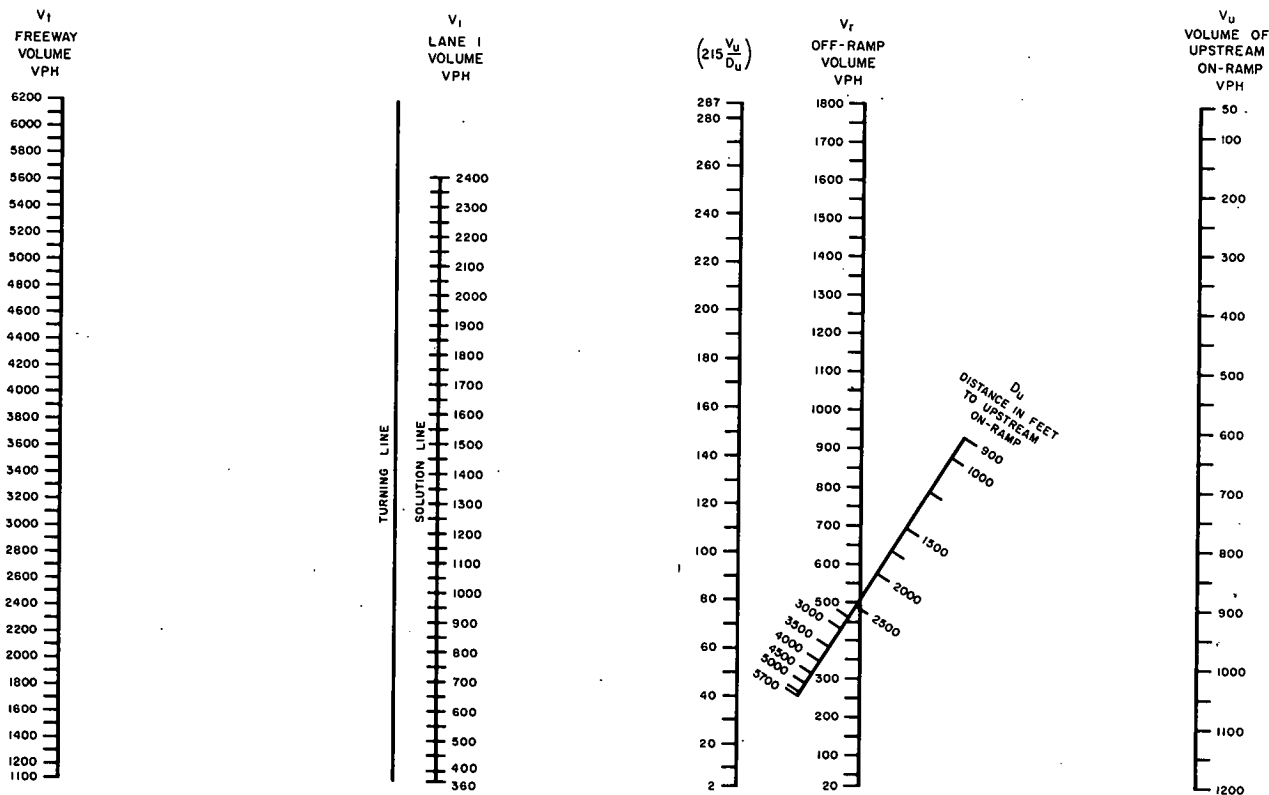
Figure 8.10. Nomograph for determination of lane 1 volume upstream of off-ramp junction, 6-lane freeway, with upstream on-ramp within 5,700 ft of off-ramp (no auxiliary lane).

V_f
UPSTREAM
FREEWAY VOLUME
V P H

SOLUTION:
V_1
UPSTREAM
VOLUME IN LANE 1
V P H

V_r
ON-RAMP VOLUME
V P H

V_d
DOWNSTREAM
OFF-RAMP VOLUME
V P H

TURNING LINE 1

SOLUTION LINE

662 VPH

(SAMPLE PROBLEM INCLUDED)

V_f
3000 VPH V_1
900 VPH V_r
V_d 400 VPH
D_d

$V_1 = 87 + 0.225\ V_f - 0.140\ V_r + 0.500\ V_d = 662$ VPH

STANDARD ERROR = 178 VPH

LIMITATIONS:

V_f : 2000 - 5600 VPH

V_r : 200 - 1500 VPH

V_d : 150 - 1500 VPH

D_d : 400' - 850'
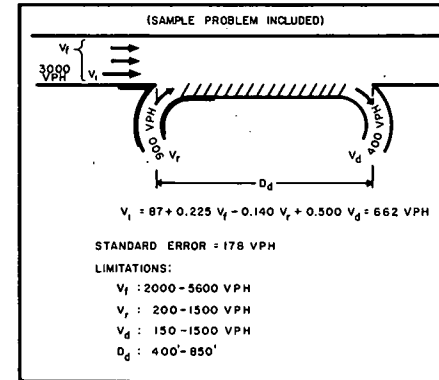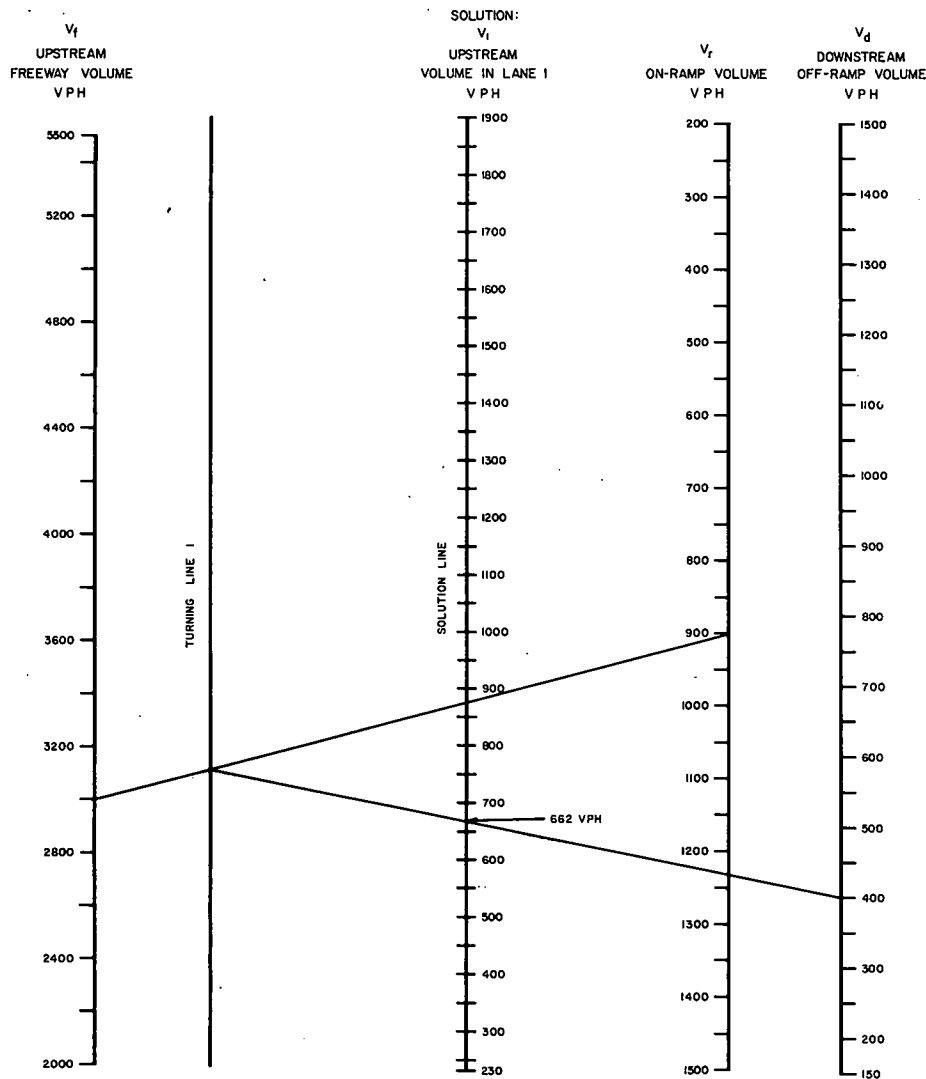
CONDITIONS FOR USE

To determine the lane 1 volume at the inner loop on-ramp nose at a cloverleaf interchange containing an auxiliary lane between the loops. The interchange may or may not have outer connections. The equation does not consider their effects and applies only to inner loop on-ramps. The inner loop off-ramp should be located between 400 and 850 ft downstream.

Reference should be made to Figure 8.20 and related discussion on auxiliary lane usage on p. 220 to check lane 1 and auxiliary lane volumes at points along the auxiliary lane. These volumes should not exceed the service volumes for the level of service involved.

STEPS IN SOLUTION

(1) Draw a line from V_f value to V_r value intersecting Turning Line 1.
(2) Draw a line from Step 1 intersection of Turning Line 1 to V_d value. The intersection point of this line with Solution Line is V_1.

Figure 8.11. Nomograph for determination of lane 1 volume upstream of on-ramp junction, 6-lane freeway, at cloverleaf inner loop with auxiliary lane.

Figure 8.12. Nomograph for determination of lane 1 volume upstream of on-ramp junction, 6-lane freeway, with auxiliary lane between on-ramp and adjacent downstream off-ramp.

V_f
FREEWAY VOLUME
UPSTREAM OF
2 ND. ON-RAMP
VPH

V_1
LANE I VOLUME
UPSTREAM OF
2 ND. ON-RAMP
VPH

V_r
2 ND. ON-RAMP
VOLUME
VPH

D_u
DISTANCE TO
UPSTREAM
ON-RAMP (FEET)

V_u
VOLUME UPSTREAM
ON-RAMP
VPH

TURNING LINE 1

SOLUTION LINE

TURNING LINE 2

V_f scale:
5400, 5000, 4600, 4200, 3800, 3400, 3000, 2600, 2200, 1800

V_1 scale:
1800, 1700, 1600, 1500, 1400, 1300, 1200, 1100, 1000, 900, 800, 700, 600, 500, 400, 300, 200, 100

V_r scale:
100, 200, 300, 400, 500, 600, 700, 800, 900, 1000, 1100, 1200, 1300, 1400, 1500

D_u scale:
500, 600, 700, 800, 900, 1000, 1100

V_u scale:
1400, 1300, 1200, 1100, 1000, 900, 800, 700, 600, 500, 400, 300, 200, 100

$V_1 = 574 + 0.228\ V_f$
$-\ 0.194\ V_r - 0.714\ D_u + 0.274\ V_u$

STANDARD ERROR = 142 VPH

LIMITATIONS :

V_f : 1800-5400 VPH
V_r : 100-1500 VPH
D_u : 500-1000'
V_u : 100-1400 VPH

CONDITIONS FOR USE

To determine the lane 1 volume at the on-ramp nose of the second of successive on-ramps.

STEPS IN SOLUTION

(1) Draw a line from V_f value to V_r value intersecting Turning Line 1.
(2) Draw a line from D_u value to V_u value intersecting Turning Line 2.
(3) Draw a line from Step 1 intersection of Turning Line 1 to Step 2 intersection of Turning Line 2. The intersection point of this line with the Solution Line is V_1 at the nose of the 2nd on-ramp.

Figure 8.13. Nomograph for determination of lane 1 volume upstream of on-ramp junction, 6-lane freeway, at second of successive on-ramps.
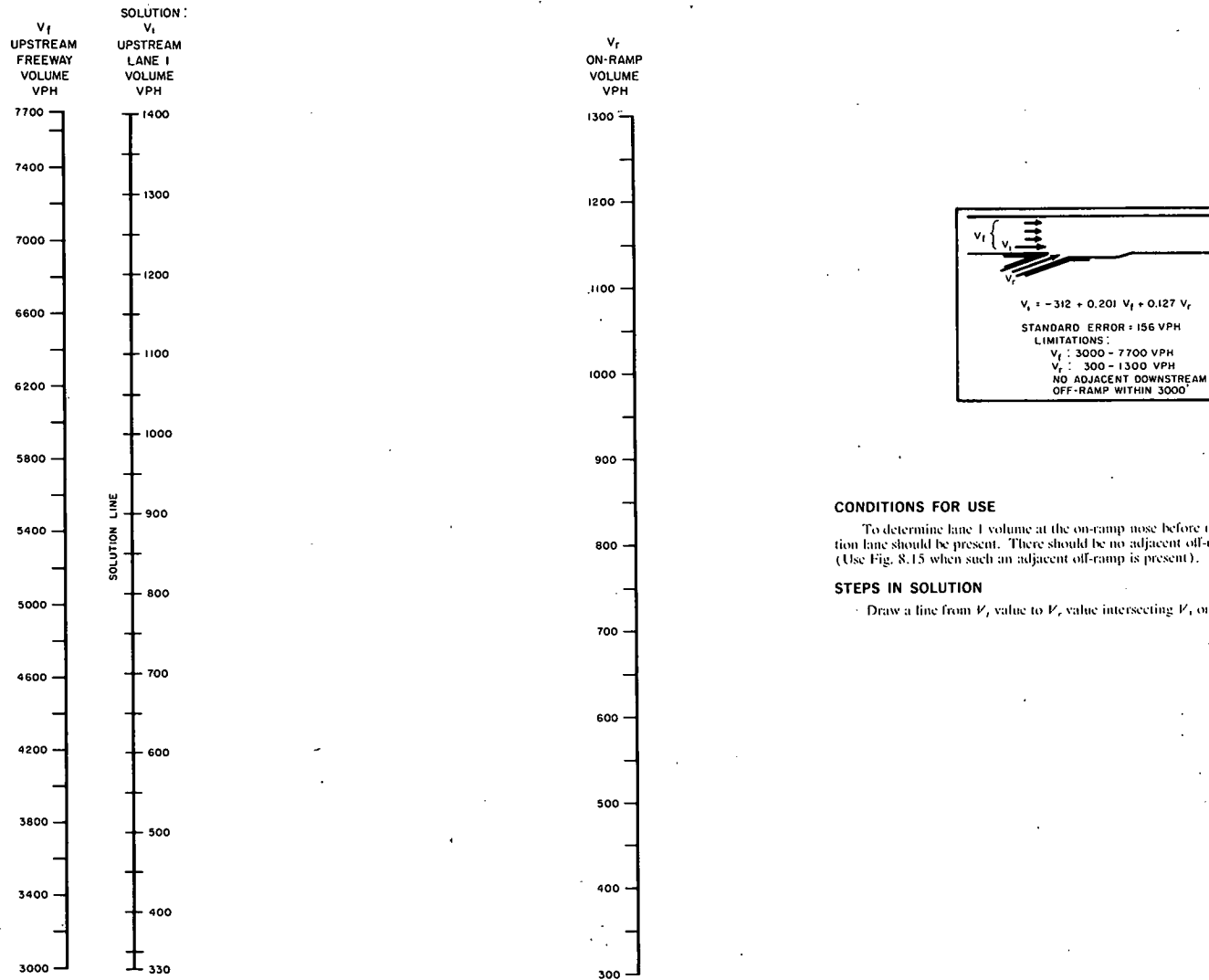
V_f
UPSTREAM
FREEWAY
VOLUME
VPH

SOLUTION:
V_1
UPSTREAM
LANE 1
VOLUME
VPH

V_r
ON-RAMP
VOLUME
VPH

7700 — 1400 — 1300
7400 — 1300 — 1200
7000 — 1200 — 1100
6600 — 1100 — 1000
6200 — 1000 — 900
5800 — 900 — 800
5400 — 800 — 700
5000 — 800 — 600
4600 — 700 — 500
4200 — 600 — 400
3800 — 500 — 300
3400 — 400
3000 — 330

SOLUTION LINE

$V_1 = -312 + 0.201\ V_f + 0.127\ V_r$

STANDARD ERROR = 156 VPH
LIMITATIONS:
V_f : 3000 – 7700 VPH
V_r : 300 – 1300 VPH
NO ADJACENT DOWNSTREAM
OFF-RAMP WITHIN 3000'

CONDITIONS FOR USE

To determine lane 1 volume at the on-ramp nose before merging takes place. An acceleration lane should be present. There should be no adjacent off-ramp within 3,000 ft downstream. (Use Fig. 8.15 when such an adjacent off-ramp is present).

STEPS IN SOLUTION

Draw a line from V_f value to V_r value intersecting V_1 on the Solution Line.

Figure 8.14. Nomograph for determination of lane 1 volume upstream of on-ramp junction, 8-lane freeway (no downstream off-ramp within 3,000 ft; no auxiliary lane).

V_f
UPSTREAM
FREEWAY
VOLUME
VPH

7100
7000

6600

6200

5800

5400

5000

4600

4200

3800

3400

3000

TURNING LINE I

SOLUTION:
V_1
UPSTREAM
LANE I
VOLUME
VPH

1400

1300

1200

1100

1000

900

800

700

600

500

400

300

230

SOLUTION LINE

V_d
VOLUME
OF ADJACENT
DOWNSTREAM
OFF-RAMP
VPH

800

700

600

500

400

300

200

100

V_r
ON-RAMP
VOLUME
VPH

300

400

500

600

700

800

900

1000

1100

$V_1 = -353 + 0.199\ V_f - 0.057\ V_r$
$+ 0.486\ V_d$

STANDARD ERROR = 107 VPH
LIMITATIONS:
V_f : 3000 – 7100 VPH
V_r : 300 – 1100 VPH
V_d : 100 – 800 VPH
D_d : 1500 – 3000'

CONDITIONS FOR USE

To determine lane 1 volume at the on-ramp nose before merging takes place. An acceleration lane should be present and there should be an adjacent downstream off-ramp 1,500 to 3,000 ft away. Thus, the distance to the adjacent downstream off-ramp D_d, is a required condition (1,500-3,000 ft), though not incorporated directly as a variable in the equation. (Use Fig. 8.14 when a downstream ramp is more than 3,000 ft away).

STEPS IN SOLUTION

(1) Draw a line from V_f value to V_r value intersecting Turning Line 1.
(2) Draw a line from Step 1 intersection of Turning Line 1 to V_d value. The intersection point of this line with the Solution Line is V_1.

Figure 8.15. Nomograph for determination of lane 1 volume upstream of on-ramp junction, 8-lane freeway, with downstream off-ramp within 1,500 to 3,000 ft (no auxiliary lane).

V_f
UPSTREAM
FREEWAY VOLUME
VPH

V_1
UPSTREAM
LANE 1 VOLUME
VPH

V_r
ON-RAMP
VOLUME
VPH

V_d
DOWNSTREAM
OFF-RAMP
VOLUME
VPH

D_d
DISTANCE TO
DOWNSTREAM
OFF-RAMP
(FEET)

TURNING LINE 1

TURNING LINE 2

SOLUTION LINE

$V_1 = 584 + 0.180 V_f - 0.203 V_r$
$\quad - 0.487 D_d + 0.204 V_d$

STANDARD ERROR = 107 VPH
LIMITATIONS:
V_f : 3670 - 7500 VPH
V_r : 110 - 1220 VPH
V_d : 110 - 1220 VPH
D_d : 500 - 1400'
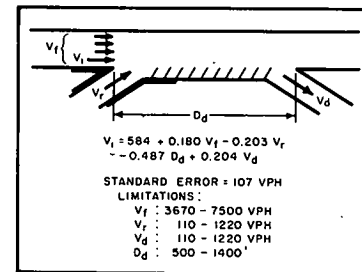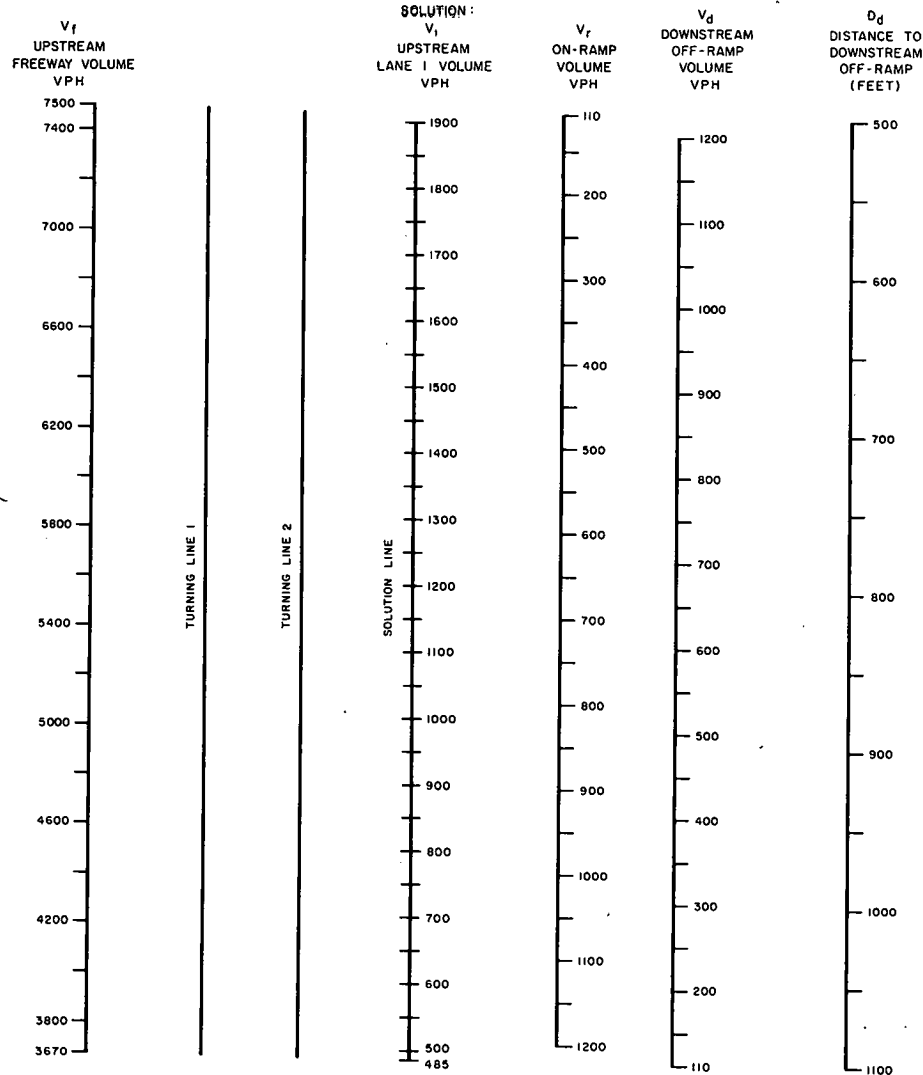
CONDITIONS FOR USE

To determine the lane 1 volume at the nose of an on-ramp where there is an auxiliary lane extending to the adjacent downstream off-ramp. The equation is primarily intended for diamond and similar flat angle entrance ramps, although it can be used in lieu of an equation for a cloverleaf loop ramp on an 8-lane freeway.

Note: Lane 1 and auxiliary lane operations should be checked by means of Figure 8.20 and related procedures on p. 220.

STEPS IN SOLUTION

(1) Draw a line from V_f value to V_r value intersecting Turning Line 1.
(2) Draw a line from Step 1 intersection of Turning Line 1 to V_d value, intersecting Turning Line 2.
(3) Draw a line from Step 2 intersection of Turning Line 2 to D_d value. The intersection point of this line with the Solution Line is V_1.

Figure 8.16. Nomograph for determination of lane 1 volume upstream of on-ramp junction, 8-lane freeway, with auxiliary lane between on-ramp and adjacent downstream off-ramp.

V_f
UPSTREAM
FREEWAY VOLUME
VPH

3000

2600

2200

1800

1400

1000

600

SOLUTION (a)
V_1
UPSTREAM
LANE 1 VOLUME
VPH

410

400

350

300

SOLUTION LINE (a)

250

200

150

SOLUTION (b)
V_{1+A}
MERGE VOLUME
(LANE 1 +
RAMP LANE A)
VPH

2380

2200

2000

1800

SOLUTION LINE (b)

1600

1400

1200

1000

800

600

V_r
TOTAL
ON-RAMP
VOLUME
VPH

3000

2800

2600

2400

2200

2000

1800

1600

1400

1200

1100

3 →
2 →
V_f { 1 }
V_1 →
V_{1+A} →
→
→
→
A
B
ACCELERATION LANE ≥ 800'
LONG REQUIRED.

SOLUTION (a)

$V_1 = 54 + 0.070 \ V_f + 0.049 \ V_r$

STANDARD ERROR = 99 VPH

LIMITATIONS:

V_f : 600 - 3000 VPH

V_r : 1100 - 3000 VPH

SOLUTION (b)

$V_{1+A} = -205 + 0.287 \ V_f + 0.575 \ V_r$

STANDARD ERROR = 212 VPH

LIMITATIONS:

V_f : 600 - 3000 VPH

V_r : 1100 - 3000 VPH

CONDITIONS FOR USE

To determine the initial merge (lane 1 + ramp lane A) and the ramp lane volume distribution at a two-lane on-ramp having an acceleration lane at least 800 ft long. There is no extra lane added to the freeway for this condition, causing ramp lane A vehicles to merge directly with lane 1 vehicles. Although sometimes used, this design is not generally recommended except under special volume conditions.

Solution (a) is used to determine V_1 at the on-ramp nose. After V_1 is calculated, Solution (b) is used to calculate the initial merge designated V_{1+A} (lane 1 + ramp lane A). The ramp lane A volume can be found by subtracting the V_1 value obtained in Solution (a) from the Solution (b) result for V_{1+A}. Ramp lane B volume is then obtained by subtracting the ramp lane A volume from V_r, the total on-ramp volume. Both V_{1+A} (lane 1 + ramp lane A) volume and the ramp lane B volume should be checked against the service volume level.

The ramp lane B vehicles use the acceleration lane and have to merge with part of the (lane 1 + ramp lane A) vehicles which have already merged. Some of the latter vehicles, however, will move into other lanes of the freeway before the ramp lane B vehicles merge with the remainder.

STEPS IN SOLUTION

(1) Draw a line from V_f value to V_r value intersecting Solution Lines (a) and (b).
(2) The intersection of Step 1 line with Solution Line (a) is V_1.
(3) The intersection of Step 1 line with Solution Line (b) is V_{1+A}.
(4) Ramp lane A volume = V_{1+A} − V_1.
(5) Ramp lane B volume = V_r − ramp lane A volume.

Figure 8.17. Nomograph for determination of lane volume distribution and merge, 6-lane freeway, at 2-lane on-ramp with acceleration lane.

SOLUTION (b)

V_t
UPSTREAM
FREEWAY VOLUME
VPH

V_1
DOWNSTREAM
LANE 1 VOLUME
VPH

SOLUTION (a)

V_{1+A}
$\begin{pmatrix} \text{LANE 1} + \\ \text{RAMP LANE A} \end{pmatrix}$
VPH

V_r
TOTAL
OFF-RAMP
VOLUME
VPH

SOLUTION (a)
$$V_{1+A} = -158 + 0.035\ V_t + 0.0567\ V_r$$

SOLUTION (b)
$$V_1 = +18 + 0.060\ V_t + 0.072\ V_r$$

STANDARD ERRORS:
(A) V_{1+A} = 181 VPH
(B) V_1 = 150 VPH
RAMP LANE A = 77 VPH

LIMITATIONS:
(BOTH EQUATIONS)
V_t : 2100-6000 VPH
V_r : 1100-6000 VPH
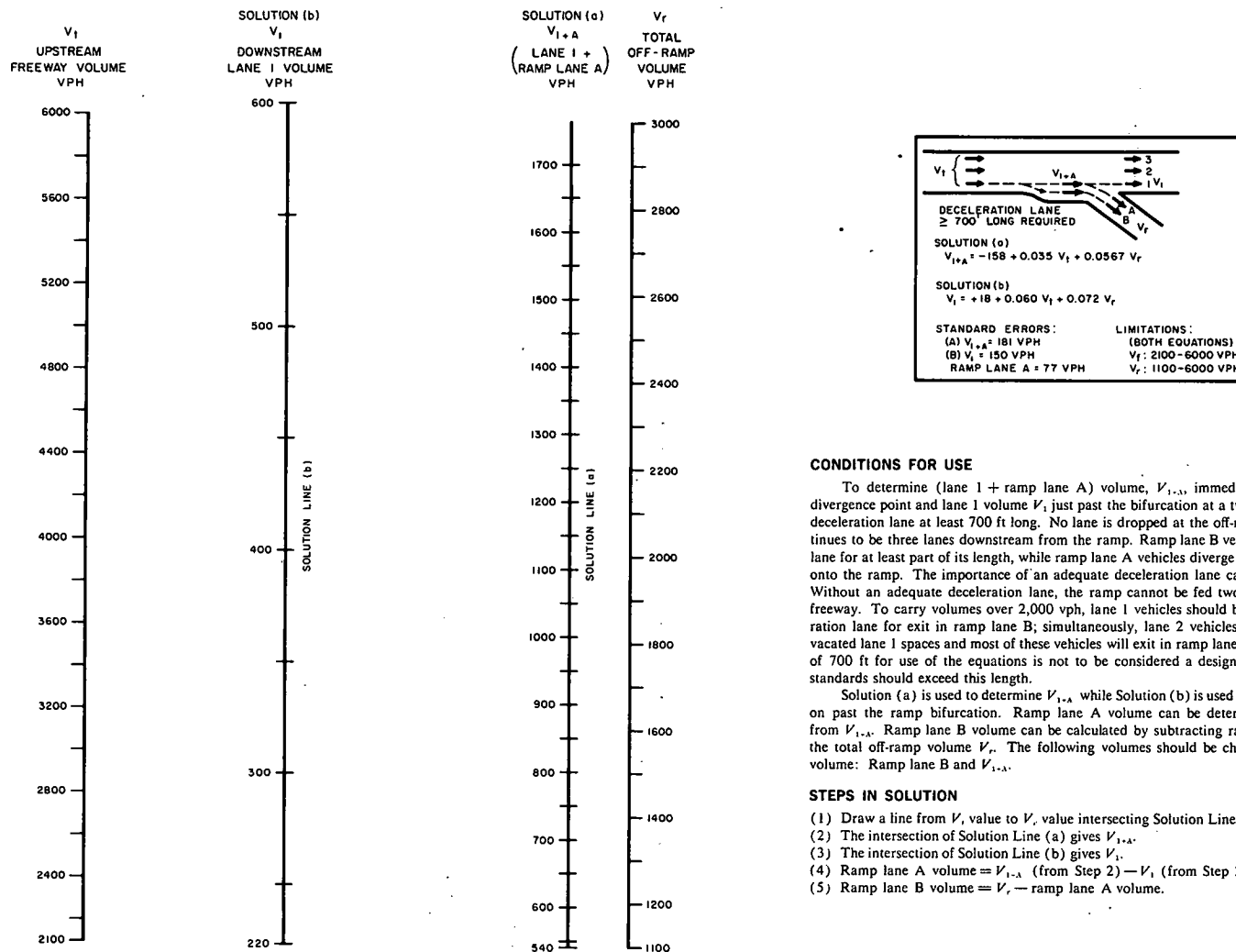
CONDITIONS FOR USE

To determine (lane 1 + ramp lane A) volume, V_{1-A}, immediately upstream from the divergence point and lane 1 volume V_1 just past the bifurcation at a two-lane off-ramp having a deceleration lane at least 700 ft long. No lane is dropped at the off-ramp, so the freeway continues to be three lanes downstream from the ramp. Ramp lane B vehicles use the deceleration lane for at least part of its length, while ramp lane A vehicles diverge directly from the freeway onto the ramp. The importance of an adequate deceleration lane cannot be overemphasized. Without an adequate deceleration lane, the ramp cannot be fed two lanes of traffic from the freeway. To carry volumes over 2,000 vph, lane 1 vehicles should be moving into the deceleration lane for exit in ramp lane B; simultaneously, lane 2 vehicles will be moving into the vacated lane 1 spaces and most of these vehicles will exit in ramp lane A. The minimum length of 700 ft for use of the equations is not to be considered a design recommendation; design standards should exceed this length.

Solution (a) is used to determine V_{1-A} while Solution (b) is used to calculate V_1 continuing on past the ramp bifurcation. Ramp lane A volume can be determined by subtracting V_1 from V_{1-A}. Ramp lane B volume can be calculated by subtracting ramp lane A volume from the total off-ramp volume V_r. The following volumes should be checked against the service volume: Ramp lane B and V_{1-A}.

STEPS IN SOLUTION

(1) Draw a line from V_t value to V_r value intersecting Solution Lines (a) and (b).
(2) The intersection of Solution Line (a) gives V_{1-A}.
(3) The intersection of Solution Line (b) gives V_1.
(4) Ramp lane A volume = V_{1-A} (from Step 2) − V_1 (from Step 3).
(5) Ramp lane B volume = V_r − ramp lane A volume.

Figure 8.18. Nomograph for determination of lane volume distribution, 6-lane freeway, upstream of 2-lane off-ramp with deceleration lane.

Column headers and scales:

V_r
TOTAL
OFF-RAMP VOLUME (A+B)
FOR SOLUTION (B)
VPH

SOLUTION (b)
V_1
DOWNSTREAM
LANE 1 VOLUME
VPH

V_f
UPSTREAM
FREEWAY VOLUME
VPH

SOLUTION (a)
V_c
(LANE 1 + LANE A)
VOLUME
VPH

V_r
TOTAL
OFF-RAMP VOLUME (A+B)
FOR SOLUTION (A)
VPH

SOLUTION LINE (b)

SOLUTION LINE (a)

Box:

SOLUTION (a)
$V_c = 64 + 0.285\,V_f + 0.141\,V_r$

SOLUTION (b)
$V_1 = 173 + 0.295\,V_f - 0.320\,V_r$

STANDARD ERRORS:

$V_c = 94$ VPH

$V_1 = 66$ VPH

RAMP LANE A VOLUME = 86 VPH

LIMITATIONS:

V_f : 1200 - 4500 VPH

V_r : 300 - 2650 VPH

CONDITIONS FOR USE

To determine lane volumes at a major fork where three lanes split into two, two-lane road-ways, with relatively balanced use of the two. Specifically, using lanes 1 and 2 to denote one roadway and lane A and lane B to denote the other roadway, the lower-volume roadway should carry at least one-third of the total volume arriving for the equation to be valid.

Solution (a) is used to calculate the center lane volume, V_c, which is subject to divergence into lanes 1 and A at the nose of the bifurcation. Solution (b) is used to determine the lane 1 volume which continues on past the nose. Knowing the total volumes of each roadway, lane 2 and lane B volumes can be calculated by subtraction. The following volumes should be checked against the service volume: V_c (lane 1 + lane A), lane 2, and lane B.

STEPS IN SOLUTION

Solution (a)

Draw a line from V_f value to V_r value on right side scale intersecting Solution Line (a). The intersection point on this line is V_c.

Solution (b)

Draw a line from V_f value to V_r value on left side scale intersecting Solution Line (b). The intersection point on this line is V_1.

Figure 8.19: Nomograph for determination of lane volume distribution, 6-lane freeway, at major fork into two 4-lane freeways.
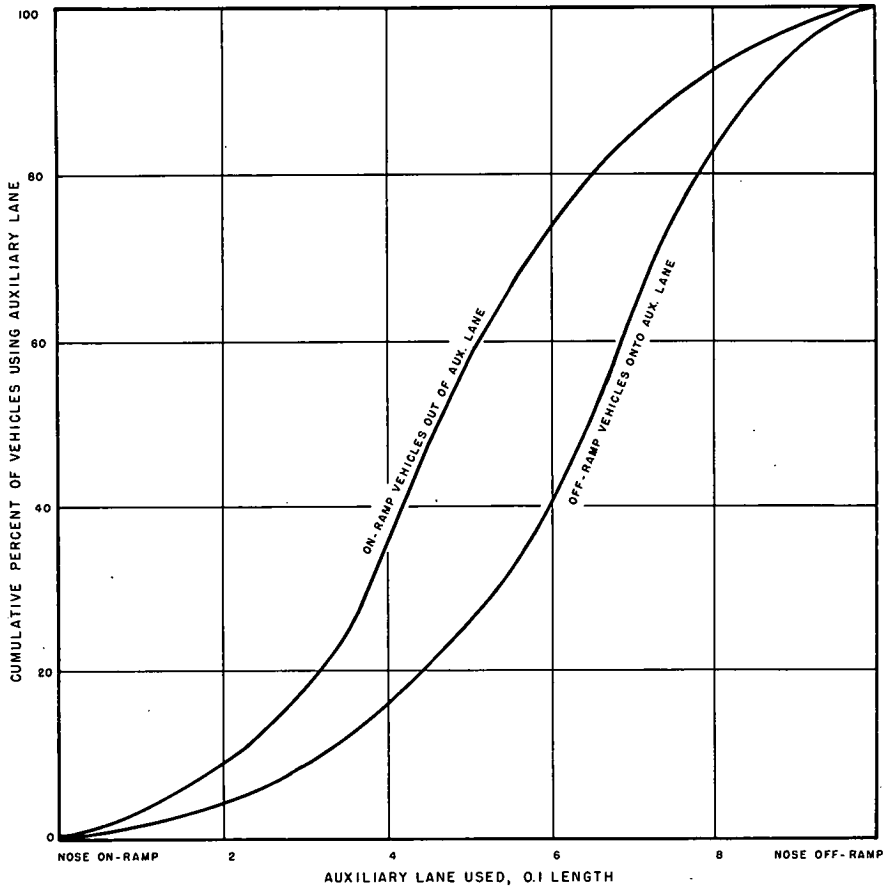
Figure 8.20. Use of auxiliary lane between adjacent on- and off-ramps.

potential overloading, as compared to the auxiliary lane which carries ramp vehicles only. The most critical checkpoint between the ramps can usually be ascertained by noting the relative ramp volumes and the shape of the curves in Figure 8.20. Examination of the upper curve in Figure 8.20 discloses that the greatest movement by on-ramp vehicles from the auxiliary lane to lane 1, over the available distance, occurs between the 0.3 and 0.6 points. Also, off-ramp vehicles tend to stay in lane 1 until the 0.5 point is reached, after which increased movement onto the auxiliary lane takes place up to the 0.8 point.

The foregoing suggest that the most

heavily traveled portion of lane 1 is the section from 0.5 to 0.6 of the distance along the auxiliary lane. As a rule of thumb, a volume check at the 0.5 point will usually suffice, where the sum of the volumes in lane 1 and the on-ramp does not exceed 150 percent of the merge service volume taken from Table 8.1, except where ramp volumes are quite high. Should the off-ramp volume be comparatively high, the lane 1 section just downstream from the on-ramp nose should be checked for overloading, say at the 0.2 point. This check is made against the merge service volume. On the other hand, if the on-ramp volume is comparatively high, a volume

check of lane 1 should be made just up-stream from the off-ramp, at possibly the 0.8 point. This check is made against the diverge service volume, because the check-point is closer to the off-ramp than to the on-ramp.

Refined Procedure for Use of Figure 8.3. —Figure 8.3 is used to determine the lane 1 volume immediately upstream from an exit ramp on 4-lane freeways if there is no adja-cent upstream on-ramp within 3,200 ft. The equation, which represents the best fit for a considerable body of data taken at 19 locations throughout the country, has a standard error of 131 vph, approximately 13 percent of the mean value of 1,022 vph for the lane 1 volume from all locations. Considering the relative variability of traffic streams, this equation may be a reasonably accurate aid as it stands. However, the large number of data makes possible increased ac-curacy by allowing a stratification of the off-ramp volumes as percentages of the freeway volume approaching the exit ramp.

Figure 8.21 is the graphical presentation of the stratified data. By following the di-verge service volume lines horizontally across the graph, the maximum percentage of al-lowable prospective off-ramp vehicles in the freeway stream can be determined. For ex-ample, at level C with a peak-hour factor of 0.83, when the maximum freeway volume of 2,500 vph is being handled, all of the ramp volume percentage lines below 40 percent ramp vehicles fall beneath the level C (0.83 PHF) line. Therefore, approximately 40 per-cent of 2,500 vph, or about 1,000 off-ramp vehicles, could be handled at the upper vol-ume limit of level C (0.83 PHF).

Multiple regression equations developed for the stratified data (2) proved quite con-clusively that the predictability of lane 1 volume upstream increases as the percentage of ramp vehicles increases. In other words, the lines representing 0-9.9 percent and 10.0-19.9 percent ramp vehicles are the least ac-curate of the five lines and the observations containing these percentages contributed more to the 131-vph standard error of the equation in Figure 8.3 than did the observa-tions having higher ramp vehicles per-centages. This is actually of advantage to the designer because the greater concern and
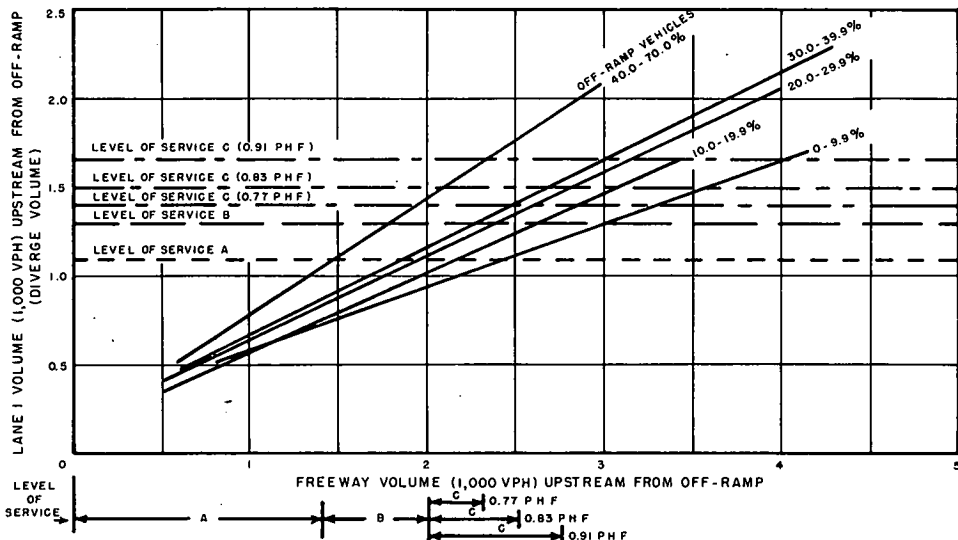
Figure 8.21. Lane 1 volume upstream from off-ramp, related to freeway volume and percentage of off-ramp vehicles in freeway stream upstream from off-ramp on 4-lane freeways (for use in conjunction with Fig. 8.3).
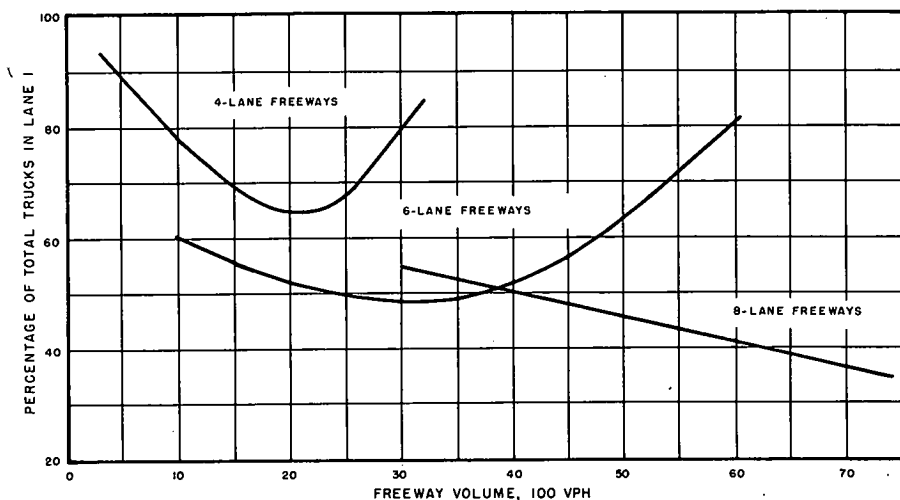
Figure 8.22. Percentage of total trucks in lane 1 of 4-, 6-, and 8-lane freeways immediately upstream from on-ramp entrances, or at the diverge point upstream from off-ramps.

need for accuracy is at the high-volume exit ramps.

Adjustment for Trucks.—The procedures that have been described are based on mixed traffic containing up to 5 percent trucks in each flow involved, in relatively level terrain. Reference to Chapter Nine shows that this range represents a truck adjustment factor range of from 1.00 for no trucks down to 0.91 for a full 5 percent. Because 5 percent trucks is taken as the base point, although no correction for less than 5 percent is required, the procedures provide a "safety factor" of up to $1/0.91 = 1.10$ where no trucks are present.

On the other hand, where more than 5 percent trucks are involved in any flow, or the grade is significant, an adjustment for trucks should be made to that flow. Figure 8.22 is provided for use in making this adjustment. It shows the percentage of trucks likely to be in lane 1 at any given freeway volume level on 4-, 6-, and 8-lane freeways. Thus, the number and percentage of trucks in V_1 can be determined, from which the passenger car equivalent and truck adjustment factor can be determined by the methods of Chapter Nine. Multiplication of the V_1 value by the factor 0.91/(actual truck adjustment factor) converts the volume to the 5 percent base inherent in the basic procedures. Similarly,

the same adjustment can be applied to ramp volumes where appropriate. This conversion permits use of the values in Table 8.1 as the fundamental set of comparison criteria, as before. Typical example 8.1, Part 2, demonstrates the computations involved.

Note: This procedure is approximate, in that the adjustments in Chapter Nine were not developed specifically for a single lane.

GEOMETRICS NOT REPRESENTED BY EQUATIONS AND NOMOGRAPHS

As previously indicated, the equations and the nomographs presented as Figures 8.2 through 8.19 do not fit all of the geometric conditions likely to be encountered in practice. This section is included, therefore, to guide the user in adapting these methods, or other procedures or references, to situations not directly covered. The designs not covered can be broken down into broad categories as follows: (1) left-hand ramps, (2) certain types of right-hand one-lane ramps, both on and off, (3) certain types of two-lane on-ramps, and (4) specialized designs sometimes needed for unusual traffic or topographic conditions. Several potential alternate methods exist for use in handling problems involving these additional geometric layouts, as well as those involving

variable values outside the ranges of values covered by the nomographs. The several possible methods, and the situations to which each is best applied, are described in this section. To the extent possible, these applications are included in the procedural index (Table 8.2).

Extrapolation of Existing Nomographs (Equations).—In cases where one or several of the variables are somewhat outside the ranges given in the nomograph, extrapolation is often acceptable, where done with caution. For example, a particular layout may involve an auxiliary lane 2,600 ft long, where the appropriate nomograph (equation) has an upper limit of 1,400 ft for length of auxiliary lane, this limit largely based on the range of data available for this condition. The equation could be used, if examination of the effect of the variable being extrapolated on the overall result of known levels indicated this was feasible. In one such case, a leveling off trend might be noted which would suggest use, in the equation, of some value short of the full 2,600 ft. In another instance, however, engineering judgment might detect an unreasonable trend which would suggest that extrapolation not be attempted.

Substitution of an Equation Representing a Relatively Similar Layout.—This is often an appropriate method, because several geometric layouts for which insufficient data are available for specialized treatment appear to operate much the same as arrangements for which data were available. In such cases, use of the figure representing the known case is recommended.

The principal such cases are as follows:

(a) One-lane on-ramp with adjacent upstream off-ramp on 4-lane freeway—Use Figure 8.2.

(b) One-lane inner loop off-ramp at cloverleaf interchange on 4-lane freeway (no auxiliary lane)—Use Figure 8.4.

(c) One-lane inner loop off-ramp at cloverleaf interchange on 4-lane freeway, with auxiliary lane—Use Figure 8.6, together with Figure 8.20.

(d) One-lane off-ramp at diamond-type interchange on 4-lane freeway, with auxiliary lane—Use Figure 8.7, together with Figure 8.20.

(e) Successive one-lane on-ramps on 4-lane freeway—For the first ramp use Figure 8.2; for the second ramp use Figure 8.8 or Figure 8.2 directly.

(f) One-lane on-ramp on 6-lane freeway, with adjacent upstream on-ramp—Use Figure 8.13 where the upstream on-ramp is within the limitation range specified. Otherwise, use Figure 8.9.

(g) One-lane inner loop off-ramp at cloverleaf interchange on 6-lane freeway, with auxiliary lane—Use Figure 8.11, together with Figure 8.20.

(h) One-lane off-ramp at diamond-type interchange on 6-lane freeway, with auxiliary lane—Use Figure 8.12, together with Figure 8.20.

(i) Successive one-lane on-ramps on 6-lane freeway—For the first ramp use Figure 8.9; for the second ramp use Figure 8.13.

(j) One-lane on-ramp with adjacent upstream off-ramp on 8-lane freeway—Use Figure 8.14 or 8.15.

(k) One-lane inner loop on-ramp at cloverleaf interchange on 8-lane freeway, with auxiliary lane—Use Figure 8.16.

Use of Procedures Presented Later in Chapter for Application to Levels of Service D and E.—Although Table 8.3 and Figure 8.24 are developed for use with the higher volumes encountered in level D, they can be applied approximately to levels B and C if no other methods appear practicable either alone or in conjunction with level B and C equations.

(a) Successive one-lane off-ramps on 4-lane freeway—For the first (upstream) off-ramp, the critical checkpoint will be lane 1 upstream of the ramp, because some of the off-ramp vehicles destined for the second of the off-ramps will be in lane 1 at the first off-ramp. Figure 8.3 or 8.4 can be used, but the off-ramp volume, V_r, used therein should be the total combined off-ramp volumes of the two off-ramps if they are closely spaced (within 800 ft of each other, nose to nose). Where the distance between ramps is between 800 and 4,000 ft, Figure 8.24b can be used to determine the number of second off-ramp vehicles in lane 1 upstream of the first. If spacing is more than 4,000 ft, conventional use of Figure 8.3 or 8.4 is recom-

mended. For the second ramp use Figure 8.3 directly.

(b) Successive one-lane off-ramps on 6-lane freeway—The procedure for the first ramp is similar to that in (a) above for 4-lane, but use Figure 8.10, together with Figure 8.24b where appropriate. For the second ramp use Figure 8.10 directly.

(c) One-lane off-ramp on 8-lane freeway—Procedures for level D, including Table 8.3 and Figure 8.24b, give approximate solutions.

(d) One-lane inner loop off-ramp at cloverleaf interchange on 8-lane freeway, with auxiliary lane—Procedures for level D, including Table 8.3 and Figure 8.24b, give approximate solutions.

(e) One-lane off-ramp at diamond-type interchange on 8-lane freeway, with auxiliary lane—Procedures for level D, including Table 8.3 and Figure 8.24b, give approximate solutions.

(f) Successive one-lane on-ramps on 8-lane freeways—Procedures for level D, including Table 8.3 and Figure 8.24a, give approximate solutions. If the upstream on-ramp has light to moderate volume (not over 600 vph), Figure 8.14 or 8.15 can be used.

(g) Successive one-lane off-ramps on 8-lane freeways—Procedures for level D, including Table 8.3 and Figure 8.24b, give approximate solutions.

Use of General Merge and Diverge Criteria in Table 8.1.

(a) Lane added to the freeway at a 1-lane on-ramp entrance, or dropped at a 1-lane off-ramp exit—The basic merge and diverge volume data in Table 8.1 may be interpreted as limiting ramp volumes.

(b) High-volume 2-lane entrance ramps, and substitutes therefor—These fall into at least four basic designs, designated as Cases I-IV as sketched in the following:

Case I—This design requires the addition

of a freeway lane and provides the outside ramp lane (lane B) with direct entry into the added freeway lane. The inside ramp lane (lane A) must merge into lane 1 of the freeway or lane B of the ramp. Research results regarding performance are not yet available; estimates are, therefore, necessary. This design approximates that of a major junction.

A suggested computation method makes use of the assumption that ramp lane B, which adds the lane to the freeway, will carry the bulk of the traffic. The amount assigned to this lane should be the merge checkpoint volume from Table 8.1. The remainder of the ramp volume should be assigned to ramp lane A; this volume will merge with the lane 1 volume of the freeway. Depending on whether the freeway upstream of the ramp has 2 lanes, 3 lanes, or 4 lanes, Figures 8.2, 8.9, and 8.14, respectively, can be applied to compute the lane 1 volume. In making the lane 1 computation, the on-ramp volume used will be that traversing ramp lane A.

For example, in a level C design for 0.83 peak-hour factor, if the total ramp volume were 1,900 vph, 1,400 vph would be assigned to ramp lane B while 500 vph would use ramp lane A and merge with the lane 1 volume of the freeway. If the freeway volume was 2,000 vph, using Figure 8.2, V_f would be 2,000 vph and V_r would be 500 vph. The computed V_1 volume added to the ramp lane A volume would be checked against the merge checkpoint volume of 1,400 vph for level C (PHF 0.83), Table 8.1.

The total downstream volume, 3,900 vph, would be similarly checked against the allowable 4,000 vph, from Table 8.1, for the given conditions.

Case II—This design also requires the addition of a freeway lane. In this case, however, the inside ramp lane (lane A) is led directly into the added freeway lane and the outside ramp lane (lane B) is expected to merge with lane A of the entrance into the
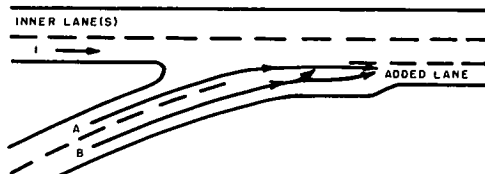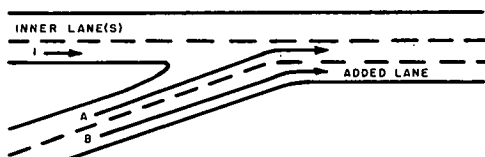
added freeway lane. Again, research results are unavailable.

A general computational method for this type of design cannot be suggested, inasmuch as marking practices can affect the paths followed by ramp drivers. Also, it is not known how many ramp lane A vehicles will move over into the adjacent left lane because of pressure exerted by ramp lane B vehicles, which must merge left into the added freeway lane. At the very least, the designer should make an "across all freeway lanes" check downstream from the merge just as he would for Case I.

Case III—This design does not require an added freeway lane but does require a long
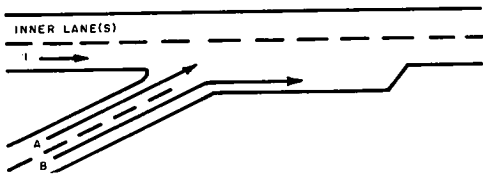
acceleration lane or reduction of lanes over a 2,000- to 3,000-ft length. Its application is mainly to the few points where upstream freeway volumes are and will remain low. Figure 8.17 applies, approximately, to the 6-lane freeway case, but research results are lacking for the 4-lane and 8-lane cases.

Case IV—This design would spread high entrance ramp volumes over two closely
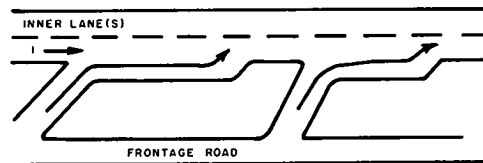
spaced ramps. It requires a frontage road or connecting surface street system. Figures 8.8 and 8.13 apply for 4-lane and 6-lane freeways, respectively, whereas Figures 8.23 and 8.24 can be used for 8-lane freeways.

Use of General Lane 1 Volume Criteria (as Found in Appendix D and Other Literature).—A variety of curves showing volume

distributions by lane have been derived; these show lane 1 volumes based on freeway volume only. Such curves can be used for general approximations.

Use of Research Literature.—Reference to the literature will show a variety of reports which can be examined for insight into this subject or even complete solutions for certain specific unusual layouts.

Left-side ramp problems, for which no generalized procedures are available for inclusion in this chapter, are discussed in two reports (9, 10) that cover extensive research into the operational characteristics (including volume distributions and merging and diverging capabilities) of several specific left-side entrance and exit ramps.

As previously indicated, only limited results of research on 2-lane ramps are available as yet. Figures 8.17 and 8.18 treat one variety each of 2-lane entrance and exit ramps on 6-lane freeways, and Figure 8.19 treats major forks, but other research results on 2-lane ramps are largely lacking. The literature should be reviewed periodically for newer, more complete findings to supplement this manual.

Local Field Sampling.—Occasionally, local on-site studies may prove the most feasible means of determining operational characteristics on existing freeways, to establish curves or approximate lane volume solutions.

Simulation by Digital Computer.—Use of digital simulation of freeway traffic flow to provide design solutions, and evaluation of alternate solutions, is becoming more convenient and feasible as programs are developed.

There remain certain highly specialized or infrequently used designs for which few general criteria can be offered at this time, due to lack of research results. Included would be designs especially fitted to the terrain or local physical conditions, major forks, and designs incorporating varying numbers of lanes. Only application of good engineering judgment, coupled with use of local studies and simulations where feasible, as just mentioned, can be suggested.

TYPICAL PROBLEM SOLUTIONS—RAMP JUNCTIONS (LEVELS A THROUGH C METHOD)

EXAMPLE 8.1

Problem:

A simple on-ramp junction exists on a 4-lane freeway (two lanes in each direction) in a medium-size city where a peak-hour factor of 0.83 has been found applicable. Traffic volumes are as shown in the sketch. Geometrics can be considered ideal.
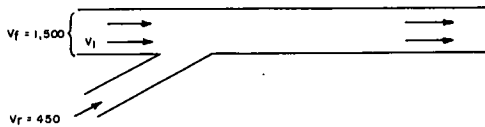
Determine (1) the level of service being provided by the junction, with no trucks or grades present; and (2) the level of service provided if the junction is located at the ½-mi point on a freeway upgrade of 3 percent, with 10 percent trucks in the freeway flow, and the ramp, carrying 4 percent trucks, is on a 4 percent upgrade ¼ mi long.

Solution:

(1) Level of service with no trucks or grades. Figure 8.2 is applicable to this design. Use of both methods, equation and nomograph, will be shown.

    (a) By use of equation:
$$V_1 = 136 + 0.345V_f - 0.115V_r$$
$$= 136 + 0.345(1,500) - 0.115(450) = 602 \text{ vph.}$$
Total merge $= V_1 + V_r = 602 + 450 = 1,052$ vph.
From Table 8.1, for PHF $= 0.83$, this merge is within level of service B.

    (b) By use of nomograph:
Enter V_f scale at 1,500 point. Draw line to 450 value on V_r scale.
Intercept with solution line is V_1 value, approx. 600 vph.
Remainder of solution same as in (a).

(2) Level of service, with substantial trucks and grades.

Ramp jct. on freeway at ½-mi point on 3 percent upgrade; ramp is ¼ mi long on 4 percent upgrade.

Truck volumes: 10 percent in freeway flow, 4 percent in ramp flow.

The procedure is carried out as before insofar as use of the equations and nomographs is concerned; no consideration is given to the trucks through that stage.

Conversion of V_1 to basic 5 percent trucks on level terrain:

From Fig. 8.22, for 1,500 vph on 4-lane freeway, 70 percent of trucks will be in lane 1, or 1,500 (0.10)(0.70) = 105 trucks in V_1.

$$\% \text{ Trucks} = \frac{\text{Trucks in } V_1}{V_1}$$
$$= 105/602 = \text{approx } 17\%.$$

From Table 9.4, for 17 percent trucks on 3 percent grade ½ mi long, $E_T = 4$.
From Table 9.6, for $E_T = 4$ and 17 percent trucks, $T_L = 0.67$.
$$V_1 = 602(0.91/0.67)$$
$$= 818 \text{ equiv. vph.}$$

Conversion of V_r to basic 5 percent trucks on level grade:

From Table 9.4, for 4 percent trucks on 4 percent grade ¼ mi long, $E_T = 10$.
From Table 9.6, for $E_T = 10$ and 4 percent trucks, $T_L = 0.74$.
$$V_r = 450(0.91/0.74)$$
$$= 553 \text{ equiv. vph.}$$

Total equivalent merge $= 818 + 553 = 1,371$ equiv. vph.

From Table 8.1 for PHF $= 0.83$, this equivalent merge is between 1,200 and 1,400, hence in level C.

EXAMPLE 8.2

Problem:

A cloverleaf interchange on a 4-lane freeway (two lanes each direction) is 400 ft long between inner loops. The total upstream freeway volume is 1,800 vph. If level B merge operation is to be maintained, what is the maximum volume that can be accommo-

dated at the on-ramp? Geometrics are adequate.
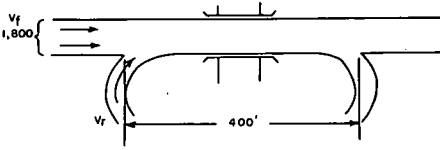
Solution:

Figure 8.5 applies; use of equation is here demonstrated.

A choice of solutions exists, depending on the general volume level of V_r.

Assume that V_r will be less than 600 vph; solution (a) applies.

$V_1 = 166 + 0.280 V_f = 166 + 0.280(1,800) = 670$ vph.

From Table 8.1, acceptable merge for level B is 1,200 vph.

$V_r = $ Acceptable merge $- V_1 = 1,200 - 670 = 530$ vph.

On-ramp can accommodate 530 vph. (This result being under 600 vph, assumption of the appropriate equation was correct. Had it been more than 600 vph, a recomputation would be required using the solution (b) equation, which gives V_r directly).

Note: Downstream operation will be in level C, inasmuch as it exceeds 2,000 vph.

EXAMPLE 8.3

Problem:

On a 6-lane freeway in a large city, where a peak-hour factor of 0.91 applies, one section has three ramp junctions in succession, the first an off-ramp, the next an on-ramp, and finally another off-ramp. Traffic estimates and geometrics are as shown in the sketch, with truck volumes under 5 percent and grades near level. Level C operation is desired.
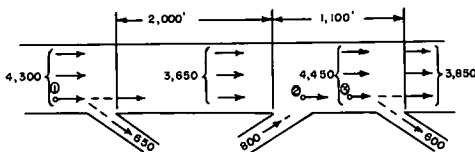
Determine (1) if the volumes shown meet level C service volumes with 0.91 PHF at the numbered checkpoints marked with a circle, (2) if the "across all freeway lanes" volume meets the freeway service volume limit set for level C and 0.91 PHF as shown for a 6-lane freeway in Table 8.1, and (3) if weaving criteria for level C are met as given in Table 8.1.

If these conditions are not met, redesign, if possible, to meet level C criteria.

Solution:

Step-by-step use of the equations will be demonstrated here, for familiarization purposes. In practice, the nomographs could be used in place of the equations for a faster graphic solution.

(a) Initial computations:

The given conditions can be evaluated at the checkpoints by use of Figure 8.10 for the diverge checkpoints and Figure 8.9 for the merge checkpoint.

At the first diverge checkpoint, using the equation in Figure 8.10,

$$V_1 = 94 + 0.231 V_t$$
$$+ 0.473 V_r + 215(V_u/D_u)$$
$$= 94 + 0.231(4,300)$$
$$+ 0.473(650) + 215(0)$$
$$= 1,395 \text{ vph.}$$

$1,395 < 1,650$, the level C diverge service volume from Table 8.1, therefore the level C service volume requirement is met and the condition is satisfactory.

At the second diverge checkpoint, using the same equation,

$$V_1 = 94 + 0.231(4,450) + 0.473(600)$$
$$+ 215(800/1,100)$$
$$= 1,562 \text{ vph.}$$

$1,562 < 1,650$; thus the level C requirement is met and the condition is satisfactory.

At the merge checkpoint, using the equation in Figure 8.9, $V_1 = -121 + 0.244 V_f - 0.085 V_u + 640 V_d/D_d = -121 + 0.244 (3,650) - 0.085(650) + 640(600/1,100) = 1,064$ vph.

Expected merge $= 1,064$ vph (V_1, at nose of on-ramp) $+ 800$ vph from on-ramp $= 1,864$ vph.

$1,864 > 1,550$, the level C merge service

volume from Table 8.1. Therefore, the merge is considerably higher than level C and does not meet the requirement. Not satisfactory.

At between-junction "across all freeway lanes" checks:

4,300 < 4,350 from Table 8.1; satisfactory.

3,650 < 4,350 from Table 8.1; satisfactory.

4,450 > 4,350 from Table 8.1; not satisfactory.

Weaving vehicles check:

800 vph "on" + 600 vph "off" = 1,400 vph weaving in 1,100 ft. This obviously meets the level C criterion of not over 1,350 vph weaving per 500 ft satisfactorily.

The foregoing analyses indicate that the proposed geometrics are deficient at the merge checkpoint, and "across the freeway lanes" between the entrance and the downstream exit ramps.

(b) Recomputations:

The most likely modification to investigate in the hope of meeting level C requirements is the addition of an auxiliary lane between the entrance and exit ramp. This will provide additional maneuvering space and reduce the number of vehicles in lane 1 between the ramps.

There is no change in operation at the first exit ramp, so no recomputation need be made there.

A volume check should be made at the nose of the on-ramp using Figure 8.12 for lane location. $V_1 = 53 + 0.283V_f - 0.402D_d + 0.547V_d = 53 + 0.283(3,650) - 0.402(1,100) + 0.547(600) = 972$ vph at the nose of the on-ramp.

The lane 1 volume at any point between the ramps will consist of through vehicles, on-ramp vehicles, and off-ramp vehicles. The number of through vehicles is determined by subtracting the off-ramp volume from the computed lane 1 volume at the nose of the on-ramp. All off-ramp vehicles are assumed to be in lane 1 at the nose of the on-ramp for computational purposes. Figure 8.20 is used to determine the lane volume distribution of the entrance and exit ramp vehicles between the ramps.

If the combined volume of the computed lane 1 volume, plus the on-ramp volume, does not exceed 150 percent of the service volume for one lane (1,550 vph for level C,

0.91 PHF), a check at the 0.5 point between the interchanges should suffice to determine if the lane 1 volume and auxiliary lane volume meet separate checks against the service volume requirement of 1,550 vph, as follows:

Lane 1 Volume Calculated at the 0.5 Point Between the Ramps:

Lane 1 through = 972 (Lane 1 calculated) − 600 (Off-ramp) = 273 vph.

On-ramp volume merged onto lane 1 = $0.58 \times 800 = 464$ vph (using Figure 8.20, upper curve).

Off-ramp volume still in lane 1 = $(1.00 − 0.25) \times 600 = 450$ vph (using Figure 8.20, lower curve).

Total in lane 1 at 0.5 point = 372 through + 464 from on-ramp + 450 destined to off-ramp = 1,286 vph.

1,286 < 1,550; lane 1 meets level C service volume requirement.

Auxiliary Lane Volume Calculated at the 0.5 Point:

The auxiliary lane volume can be calculated using Figure 8.20, but it is perhaps easier to simply add the lane 1 volume at the nose to the on-ramp volume and subtract the volume calculated to be in lane 1 at the 0.5 point.

972 (at nose) + 800 (on-ramp) − 1,286 (lane 1 at 0.5 point) = 486 vph.

486 < 1,550; auxiliary lane meets level C service volume requirement.

There is also the need to check the "across all freeway lanes" volume to make sure it now meets the 4,350 vph requirement. In such checks, the auxiliary lane is not counted as a lane and the volume carried on the auxiliary lane at the checkpoint is not counted in the freeway volume.

In this problem, there are 4,450 vph entering the section and 486 vph have been calculated to be in the auxiliary lane at the 0.5 point. There remain 3,964 vph on the three through lanes.

3,964 < 4,350; "across all freeway lanes" requirement for level C is met.

A final check can be made to assure that the service volume requirement for weaving is met. Inasmuch as the weaving criterion is 1,350 vph per 500 ft of roadway for level C (0.91 PHF), the weaving of 1,400 vph (800

+600) in 1,100 ft appears satisfactory. A check of the ramp volumes already calculated to have moved out of their "entering the section" lanes and to have completed their weave, shows 464 on-ramp vehicles have weaved with 150 off-ramp vehicles, for a total weave of 614 vph over the first 550 ft (0.5 of 1,100 ft) of the weaving section. This is considerably less than the 1,350 vph upper limit for 500 ft of roadway as given in Table 8.1.

The revised geometrics, shown with expected volumes in the following sketch, now meet level C specifications.
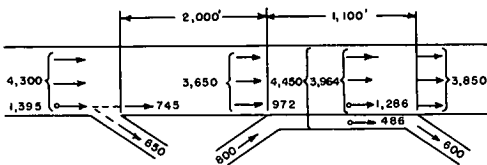
EXAMPLE 8.4

Problem:

Six-lane freeway with geometrics and volumes as shown in the sketch.
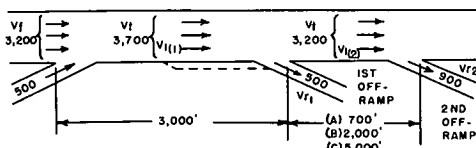
Given: Level C; peak-hour factor = 0.83. Truck volumes under 5 percent, grades under 2 percent.

Determine: Whether or not the diverge volumes (V_1) upstream of the off-ramps meet level C requirements for the following three conditions of distance between off-ramp noses: (a) 700 ft, (b) 2,000 ft, (c) 5,000 ft.

Solution:

Figure 8.10 applies, in conjunction with Figure 8.23 in some cases. Use of the equations is here demonstrated; nomographic solution would reduce the computational time.

(a) For 700 ft between off-ramp noses:

Where off-ramps are 800 ft or less apart, the ramp vehicles destined for the second off-ramp are assumed to be in lane 1 upstream of the first off-ramp.

First off-ramp:

$V_1 = +94 + 0.231 V_t + 0.473(V_{r_1} + V_{r_2}) + 215 V_u/D_u = +94 + 0.231(3,700) + 0.473(500 + 900) + 215(500/3,000) = 1,647$ vph. 1,647 > 1,500, the level C diverge service volume from Table 8.1 for PHF = 0.83, therefore the service volume requirement is not met.

Second off-ramp:

No calculation is needed. The requirement will be met if the first off-ramp requirements are met.

(b) For 2,000 ft between off-ramp noses:

Where off-ramps are 800 to 4,000 ft apart, Figure 8.23b is used to determine the number of vehicles destined for the second off-ramp which will be in lane 1 upstream of the first off-ramp.

First off-ramp:

From Table 8.23b, 63 percent of the second off-ramp vehicles will be in lane 1 at a point 2,000 ft upstream of that ramp.

$V_1 = +94 + 0.231 V_t + 0.473(V_{r_1} + 0.63 V_{r_2}) + 215(V_u/D_u) = +94 + 0.231(3,700) + 0.473[500 + 0.63(900)] + 215(500/3,000) = 1,489$ vph.

1,489 < 1,500, therefore the level C requirement is met.

Second off-ramp:

$V_1 = +94 + 0.231 V_t + 0.473 V_{r_2} + 215 V_u/D_u = +94 + 0.231(3,200) + 0.473(900) + 215(500/5,000) = 1,280$ vph.

1,280 < 1,500, therefore the level C requirement is met.

(c) For 5,000 ft between off-ramp noses:

Where off-ramps are more than 4,000 ft apart, the second off-ramp can be considered to have no effect on the first off-ramp. Also, an on-ramp upstream of the first off-ramp would be disregarded in calculations for the second off-ramp.

First off-ramp:

$V = +94 + 0.231 V_t + 0.473 V_{r_1} + 215 V_u/D_u = +94 + 0.231(3,700) + 0.473(500) + 215(500/3,000) = 1,221$ vph.

1,221 < 1,500, therefore the level C requirement is met.

Second off-ramp:

$$V_1 = +94 + 0.231V_t + 0.473V_{r_2} + 215V_u/$$
$$D_u = +94 + 0.231 (3,200) + 0.473 (900) + 215(0) = 1,259 \text{ vph.}$$

1,259 < 1,500, therefore the level C requirement is met.

The maximum between-junction "across all freeway lanes" volume check is satisfactory; 3,700 < 4,000 vph, limit for level C, 0.83 PHF.

Weaving vehicles check:

The presence of one entrance and two exits indicates that there will be at least rudimentary multiple-weaving characteristics. Inasmuch as the on-ramp carrying 500 vph is 3,000 ft upstream, it is obvious that the level C requirement of less than 1,200 vph weaving in any 500 ft segment is met.

(d) Conclusions:

The several solutions above have shown that with the given volumes, level C service volume requirements are not met upstream of the first off-ramp when only 700 ft separate the off-ramps. For the other distances of 2,000 and 5,000 ft, the design is satisfactory.

(e) Alternates to consider, in 700-ft case:

There are several alternates to consider for the situation where only a 700-ft distance exists between off-ramp noses.

(1) Accept level of service D at the first off-ramp.

(2) Separate the off-ramps by a greater distance. The solution for 2,000-ft distance is shown to barely meet level C requirements. The off-ramps, then, should be at least 2,000 ft apart.

(3) Provide a long parallel deceleration lane upstream of the first off-ramp so that the 500 vph destined for the first off-ramp can be off the through lanes before the majority of the 900 vph destined for the second off-ramp have moved over into lane 1. By use of Figures 8.10 and 8.23b it can be determined that the deceleration lane should start at least 1,600 ft upstream of the first off-ramp. Assuming that the 500 vph will move onto the deceleration lane along the first 600 ft of its length, a lane 1 check should be made 1,000 ft upstream of the off-ramp and also 1,600 ft upstream to check utilization of lane 1 just before the deceleration lane begins.

(4) Provide an auxiliary lane between the upstream on-ramp and the first off-ramp. Obviously, this should remedy the situation; but if there is doubt, Figure 8.12 and 8.20 should be used to analyze lane 1 volumes at several points between the on-ramp and the off-ramp. (From a capacity standpoint, a continuation of this lane to the second ramp probably would be found unnecessary; from a safety standpoint, it might be found desirable, however).

EXAMPLE 8.5

Part a.

Problem:

Given: A two-lane on-ramp joins a 6-lane freeway, and at the point of junction a lane is added to form an 8-lane freeway downstream (see sketch). Level of service C is desired, and PHF = 0.91; geometrics are adequate. The freeway is carrying 4,000 vph approaching the on-ramp. The truck percentage is under 5 and grades are near level.
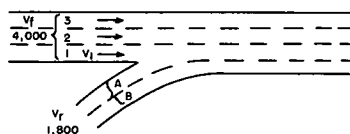
Determine: (1) Will the assigned volumes as shown in the sketch meet level C requirements? (2) What is the maximum allowable ramp volume for level C?

Solution:

(1) Check of assigned traffic volumes relative to level C:

From Table 8.1, the allowable upstream freeway volume for a 6-lane freeway at level C with PHF = 0.91 is 4,350 vph, whereas the allowable downstream freeway volume for an 8-lane freeway at level C with PHF = 0.91 is 6,000 vph. The layout thus meets "across all freeway lanes" checks, for level C.

This is a 2-lane on-ramp, Case I design, as discussed on p. 226, because the freeway lane is added as a continuation of the right

ramp lane (ramp lane B). Because ramp lane B has free access to the freeway, it is assumed to carry the bulk of the volume and is assigned 1,550 vph, the merge checkpoint volume for level C, PHF = 0.91, from Table 8.1. The remainder of the ramp traffic (1,800 − 1,550 = 250 vph) is assumed to merge with freeway lane 1 traffic, much as it would in an ordinary merge (without an auxiliary lane). Therefore, in order to find the expected merge, Figure 8.9 can be used to find V_1 for the 6-lane freeway.

$V_1 = -121 + 0.244V_f - 0.085V_u + 640V_d/$
$D_d = -121 + 0.244(4,000) - 0.085(0) + 640$
$(0) = 855$ vph.

Total merge = V_1 + Ramp lane A volume = 855 + 250 = 1,105 vph.

1,105 < 1,550, therefore the merge is satisfactory for level C, PHF = 0.91, from Table 8.1 and the assigned ramp volume of 1,800 vph meets level C requirements.

(2) Maximum allowable ramp volume at level C on freeway for given upstream volume:

The maximum allowable freeway volume is 6,000 vph downstream of the merge, from Table 8.1, for 8-lane freeway, level C, PHF = 0.91. Therefore, the maximum allowable ramp volume, based on through freeway capabilities, is

$V_r = V_f$ (allowable downstream) − V_f (actual upstream) = 6,000 − 4,000 = 2,000 vph.

Checking the merge in lane 1,

Total merge (vph) = V_1 + Maximum possible ramp lane A volume = 855 + (2,000 − 1,550) = 1,305 vph.

1,305 < 1,550, therefore the merge is satisfactory, but the maximum allowable ramp volume under level C is 2,000 vph as constrained by the allowable freeway volume downstream.

Part b.

Problem:

Given: Same layout and level of service requirements as in Part (a). However, the freeway is carrying 3,200 vph approaching the on-ramp.

Determine: The allowable ramp volume under level C.
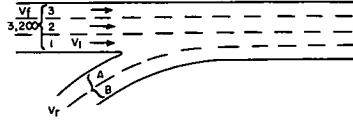
Solution:

The ramp could conceivably carry 6,000 − 3,200 = 2,800 vph based on "across all freeway lanes" volumes. However, the merge of ramp lane A with V_1 of the freeway must be checked for these new conditions.

$V_1 = -121 + 0.244V_f - 0.085V_u + 640V_d/$
$D_d = -121 + 0.244(3,200) - 0.085(0) + 640(0) = 660$ vph.

Total merge = V_1 + Ramp lane A volume = 660 + (2,800 − 1,550) = 1,910 vph.

1,910 > 1,550, therefore the merge is not satisfactory.

The maximum allowable ramp volume to meet the level C requirement of 1,550 vph merge is obtained as follows:

Maximum allowable merge = V_1 + (V_r − 1,550 in lane B)

1,550 vph merge = 660 + (V_r − 1,550)

V_r = 2,440 vph, allowable ramp volume at level C.

Evaluation:

Thus, the freeway downstream volume was the constraint in part (a), whereas the ramp volume was the constraint in part (b) where the freeway and ramp volumes are more nearly equal.

Calculation of Service Volumes, Levels D and E (Capacity)

DIFFERENCES FROM BETTER LEVELS

The preceding section of this chapter considers operations at ramp-freeway junctions at various service volumes providing free flow. Equations and nomographs presented in that section, although based on observations of flow at all levels, are directed particularly toward level C, which represents merge volumes of 1,300 to 1,550 vph during a whole hour, implying short-term (5-min) peak rates of 1,700 vph. Under some conditions of relative traffic movements (on, off, and through), these might approach the maximum values for stable flow.

In most cases, however, on reasonably well-designed freeways, stability extends well into level D, and it is in this area that the practicing traffic engineer is most interested. Desirable though it would be to operate all highways at level C or better, this cannot yet be done in most cities. As a result, the traffic engineer is forced to think in terms of "how many can I consistently get through" in many peak-period situations.

Unlike mainline flow, there is a considerable range in workable ramp junction volumes within level D, before level E is reached. This is true because mainline traffic not only distributes itself (by lanes) differently under conditions of incipient congestion, but also distributes itself in a variety of ways depending on the specific combination of geometrics at any particular site. The criteria in Table 8.1 show the upper limit of merging or diverging volume (or volume in any one lane) as 2,000 passenger cars in a whole hour, but the lane distribution changes result in relatively less traffic in lane 1, and more in the remaining lanes. It is therefore possible with this type of operation to obtain greater ramp volumes than those obtained at the high-volume end of level C.

As an example, suppose that there are 3,000 vph on a 6-lane freeway upstream of an on-ramp with geometrics as shown in Figure 8.9. The adjacent ramps are an upstream off-ramp carrying 400 vph and an off-ramp 4,000 ft downstream carrying 500 vph. The nomographic solution (Fig. 8.9) shows that 660 vph of the 3,000 vph are in lane 1 and at the upper volume limit of level C only 640 vph (for 0.77 PHF), 740 vph (for 0.83 PHF), or 890 vph (for 0.91 PHF) can enter from the on-ramp at this location without exceeding the merge limits. If the actual demand at the on-ramp happens to be at these volumes or slightly lower, this is a true solution, and merging will take place at level C. However, it will be seen that the average lane volume in lanes 2 and 3 under these stipulated conditions would be only $(3,000-660)/2=1,170$ vph. If the on-ramp volume is greater than the 640 vph, 740 vph, or 890 vph stated above, queuing in the right lane will be incipient, but queues will not form because there is so much room in lanes 2 and 3 that some drivers in lane 1 will

change lanes before suffering reduced speed or stoppage. This is particularly true when most of the drivers are repeat users, or "commuters," as is the case at most locations where capacity is a problem.

The procedures previously described for levels A through C can be applied to level D and, approximately, to level E, in the same way that they are applied to the better levels, through selection of the appropriate control values from Table 8.1. Occasionally, for certain special geometric situations, this may be necessary. However, other procedures are available, for most typical geometric arrangements, which apply specifically to the level D case; they can be used to approximate level E also (3). These are next described.

COMPUTATIONAL PROCEDURES—LEVEL D

Table 8.3 and Figure 8.23 are the principal computational devices reflecting typical driver behavior at level of service D. Table 8.3 gives the percentage of through traffic likely to remain in lane 1 through the ramp junction area at level D on 4-, 6-, and 8-lane freeways. Similarly, Figure 8.23 shows the percentages of on-ramp and off-ramp traffic likely to be in lane 1 (as well as in the auxiliary lane, if one is present) through the same area, on freeways of any normal number of lanes. It is derived from Figures 8.24 and 8.25.

Workable ramp volumes at the high-volume end of level of service D can be determined by means of procedures making use of Table 8.3 and Figure 8.23. Allowance has been made for peaking within the hour, and for normal variation in other unmeasured conditions. If the check point volumes do not exceed those associated with level D, capacity will seldom, if ever, be exceeded; hence, queues will not form. These values, then, represent the highest volumes that can be consistently carried with little likelihood of a flow "breakdown." Conditions, however, may seem restricted to many drivers.

This procedure and these values should be used to check operational problems on existing freeways and can be used in design to check critical locations to ensure that they will not become bottlenecks which would affect level of service at upstream locations.

Briefly the procedural steps are as follows:

(a) Establish the geometrics of the location under study, including number of freeway lanes, location of all ramps within a distance of 4,000 ft upstream and downstream of the ramp or point under study, and the existence or absence of auxiliary lanes.

(b) Establish the demand volumes for all movements involved.

(c) Determine flow by lanes in the section under consideration at critical points such as shown in Figure 8.1 and at 500-ft intervals through the critical section, using Table 8.3 and Figure 8.23. Check these flows against control values, as follows:

(1) The merge volume in lane 1 or the auxiliary lane at any point cannot exceed the value listed in Table 8.1 for the level of service selected; it ranges from 1,400 to 1,650 vph for level D, depending on the peak-hour factor selected. Similarly, the diverge volume (total volume before splitting into through and off flows) at a point in lane 1 or the auxiliary lane just upstream of an off-ramp cannot exceed the value listed in Table 8.1 for the level selected; for Level D this ranges from 1,500 to 1,750 vph, also depending on peak-hour factor. The volume at critical locations, such as shown in Figure 8.1, should be checked against these values. Inspection of Figure 8.23 will assist in determining where critical points exist.

(2) The total volume across all freeway lanes (excluding an auxiliary lane, if one is present, and the volume in that lane) should not exceed the limiting volume given in Table 8.1 for the level of service desired on the mainline freeway, if balanced flow is to be maintained.

(3) The number of weaving vehicles should not exceed from 1,400 to 1,650 vph on any 500-ft segment of a weaving section (again depending on the peak-hour factor selected, as given in Table 8.1).

(d) Evaluate the results of the examination in Step (c). If unsatisfactory, consider possible corrective measures.

Step (c), as listed above, is the principal determination step in this procedure; it in-

TABLE 8.3—APPROXIMATE PERCENTAGE OF THROUGH[a] TRAFFIC REMAINING IN LANE 1 IN THE VICINITY OF RAMP TERMINALS AT LEVEL OF SERVICE D

TOTAL VOLUME OF THROUGH TRAFFIC, ONE DIRECTION (vph)	THROUGH TRAFFIC REMAINING IN LANE 1 (%)		
	8-LANE[b] FREEWAY	6-LANE[c] FREEWAY	4-LANE[d] FREEWAY
6500 and over	10	—	—
6000–6499	10	—	—
5500–5999	10	—	—
5000–5499	9	—	—
4500–4999	9	18	—
4000–4499	8	14	—
3500–3999	8	10	—
3000–3499	8	6	40
2500–2999	8	6	35
2000–2499	8	6	30
1500–1999	8	6	25
Up to 1499	8	6	20

[a] Traffic not involved in a ramp movement within 4,000 ft in either direction.
[b] 4 lanes one way.
[c] 3 lanes one way.
[d] 2 lanes one way.

volves use of Table 8.3 and Figure 8.23. A clear understanding of this step requires knowledge of the derivation of Figure 8.23, which is based principally on Figures 8.24 and 8.25. Figure 8.24 represents Case I, where no auxiliary lane is present; Figure 8.25 represents Case II, where an auxiliary lane exists.

Considering Case I first, Figure 8.24a applies to on-ramps without a downstream auxiliary lane. It indicates the probable volume of on-ramp traffic in the right lane at any point downstream of the on-ramp. For example, 500 ft downstream of the on-ramp nose, 100 percent of the ramp traffic will have at least encroached on the right-hand freeway lane (lane 1). The whole vehicle may not yet be in the right lane, but the left side will be close enough to create a headway unit in it. Downstream 1,000 ft from the

CASE I – SINGLE-LANE ON- AND OFF-RAMPS WITHOUT AUXILLIARY LANE
(THIS CHART MAY BE USED REGARDLESS OF ACTUAL SPACING BETWEEN
ON- AND OFF- RAMPS, BUT AS NOTED BELOW* CAUTION MUST BE
EXERCISED IN USING THESE VALUES.)
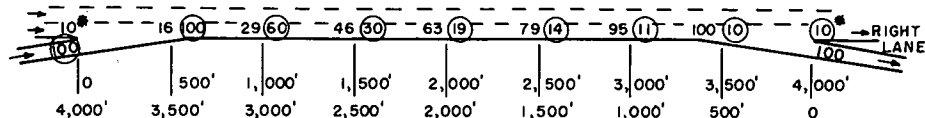
CASE II – SINGLE-LANE ON- AND OFF-RAMPS WITH AUXILLIARY LANE**
(A) L (LENGTH OF AUX. LANE BETWEEN NOSES) = 1,000'

EXAMPLE OF USE OF FIGURE 8.23
(SUMMARY OF EXAMPLE 8.6)

GIVEN: L = 1,000'
PORTION OF V₁ THROUGH (FROM
TABLE 8.3) = 475 VPH
ON-RAMP = 1,000 VPH
OFF-RAMP = 1,200 VPH
ON-RAMP TO OFF-RAMP = 0

FIND: V_1 (VOL. IN LANE 1) @ 500' =
475 + (0.80)(1,000) + (0.24)(1,200) =
1,563 VPH
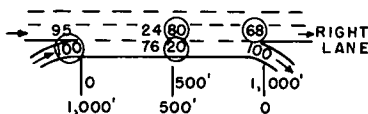
(B) L = 1,500'
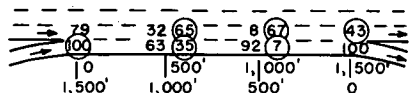
(C) L = 2,000'
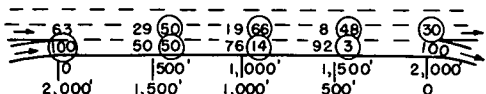
(D) L = 2,500'
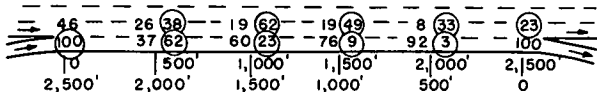
(E) L = 3,000'
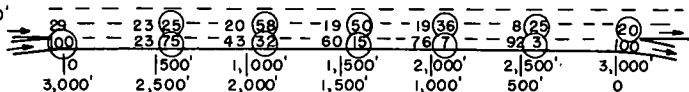
CIRCLED VALUES Ⓞ INDICATE PERCENTAGE OF ON-RAMP TRAFFIC IN LANE SHOWN. UNCIRCLED
VALUES INDICATE PERCENTAGE OF OFF-RAMP TRAFFIC IN LANE SHOWN. (REMAINING PORTION OF
TRAFFIC IS IN LANE(S) TO LEFT OF LANE 1.)

THESE PERCENTAGES ARE NOT NECESSARILY THE DISTRIBUTIONS UNDER FREE FLOW OR
LIGHT RAMP TRAFFIC, BUT UNDER PRESSURE OF HIGH VOLUMES IN THE RIGHT LANES AT THE
POINT BEING CONSIDERED AND WITH ROOM AVAILABLE IN OTHER LANES.

* MINIMUM % IN RIGHT LANE CANNOT BE LESS THAN % OF THROUGH TRAFFIC IN RIGHT
LANE AS DETERMINED FROM TABLE 8.3 (SEE NOTE, FIG. 8.24).

** SEE FIGURE 7.5 FOR METHOD OF MEASURING LENGTH L.

Figure 8.23. Percentage distribution of on- and off-ramp traffic in lane 1 and auxiliary lane.

Figure 8.24. Percentage of ramp traffic in lane 1 (no auxiliary lane).

nose 60 percent will be in the right lane, with the other 40 percent having moved over to the left if there is room in the other lane and if there is a reason to avoid lane 1 (such as a downstream merge conflict).

Figure 8.24b applies to off-ramps not preceded by an auxiliary lane. It indicates the average volume of off-ramp traffic in the right lane (lane 1) at any distance upstream of the ramp nose. It is shown, for example, that in the case of a conventional off-ramp (no auxiliary lane, standard taper) 100 percent of the off-ramp traffic will be in the right lane (lane 1) at a point 500 ft upstream of the off-ramp nose. At a point 2,000 ft upstream of the nose 63 percent of the off-ramp traffic will still be in lane 2, provided there is a reason (congestion) to stay out of lane 1 as long as possible yet there is room to move into lane 1 downstream before the off-ramp is reached.

This figure illustrates an important point in connection with an ordinary off-ramp. Inasmuch as there is always some through traffic in the right lane, it would never be possible to supply the capacity volume of a full lane to an off-ramp even though the ramp itself might be able to accommodate it. If a parallel auxiliary lane is added, however, the capacity of a full lane can be supplied to a ramp. Thus, under high volume conditions provision of a parallel lane increases the capacity of off-ramps although a simple taper is adequate under lower volume conditions.

A typical example of use of the charts is included in Figure 8.24; it involves a check of operation at a point 1,500 ft downstream of an on-ramp entering a 4-lane freeway (2 lanes in each direction).

At locations classed as Case II, involving an on-ramp connected by an auxiliary lane to a downstream off-ramp, distributions of on-ramp traffic differ; typical distributions are shown in Figure 8.25. For example, the

1,000' DOWNSTREAM OF NOSE 86% OF 1,200 ON-RAMP
VEHICLES WILL BE OUT OF THE AUXILIARY LANE.
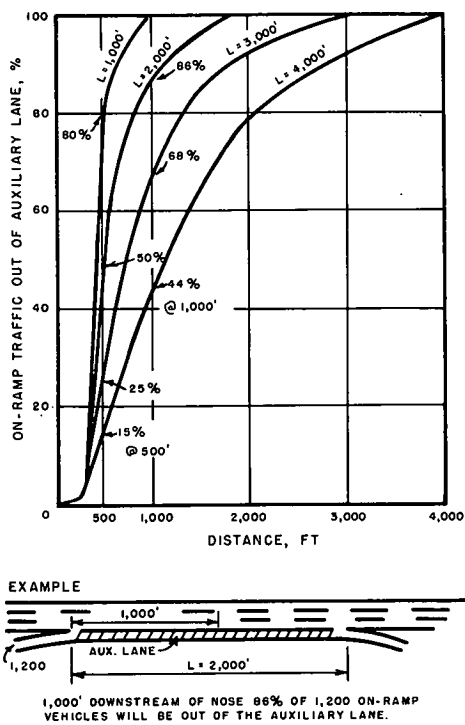
Figure 8.25. Percentage of on-ramp traffic leaving auxiliary lane at any point for a given length of auxiliary lane, L.

figure indicates that if a 2,000-ft auxiliary lane is provided, 86 percent of the on-ramp traffic will be out of the auxiliary lane 1,000 ft downstream of the on-ramp.

For general computational use, Figure 8.23 combines data from these several figures. It shows, for several representative distances between ramp junctions, including both locations with and without auxiliary lanes, the distribution of ramp traffic at 500-ft intervals. That is, it shows the percentages of both the traffic entering from the on-ramp and that preparing to exit at the off-ramp which will be found in the auxiliary lane and in lane 1 at these 500-ft points. (Although not required for computational purposes, the difference between the total of the percentages in lane 1 and the auxiliary lane and 100 percent represents on-ramp traffic that has reached lane 2, or off-ramp

traffic that has not yet moved right from lane 2.)

In measuring the distances between noses, again the method outlined in Figure 7.5 should be used. Obviously, in actual practice there are few weaving sections whose lengths are exact multiples of 500 ft; however, the length of the section under investigation can be rounded to the nearest 500 ft for the purpose of using Figure 8.23 without exceeding allowable error in estimating the acceptability of traffic operation.

In carrying out procedural step (c) to determine lane 1 volumes in Case I where no auxiliary lane is present, Table 8.3 is first used to determine the percentage of through traffic (defined for purposes of this section as traffic not involved in a ramp movement within 4,000 ft in either direction) that will probably remain in lane 1 throughout the entire merging and weaving section, at level D. The total directional through volume is then multiplied by the percentage just selected from Table 8.3 to obtain the through (non-weaving) volume in lane 1. Next, to this volume is added the on-ramp traffic which has entered lane 1, as well as the off-ramp traffic which has not yet left lane 1, as determined from Figure 8.23a. The sum is the total volume in lane 1 at the particular point.

In Case II where there is an auxiliary lane, Figure 8.23b is employed in a similar manner. Here, however, the auxiliary lane volumes, consisting of on-ramp vehicles still in that lane plus off-ramp vehicles which have already entered it, must also be determined.

In either case, as previously described the lane volumes obtained must be checked against control values.

The method can be utilized for a wide variety of geometrics, as is demonstrated in the sample problems which follow. In particular, it can be adapted to cases where a pair of on- or off-ramps exists, as shown in Example 8.9. In such cases, the flow to or from the outermost ramp of the pair should be examined for its influence on lane 1, both as an individual ramp flow and as part of the through flow past the next ramp; the result selected should be that producing the largest volume contribution to lane 1.

TYPICAL PROBLEM SOLUTIONS—RAMP JUNC-
TIONS (LEVEL D METHOD)

EXAMPLE 8.6 (Showing use of Fig. 8.23)

(Note: Figure 8.23 incorporates a sum-
mary of this simple example of the basic
procedure covered in Case II of the figure.)
Problem:

As shown in the figure, assume a 4-lane
freeway (two lanes in each direction) having
on- and off-ramps 1,000 ft apart with an
auxiliary lane between them, a peak-hour
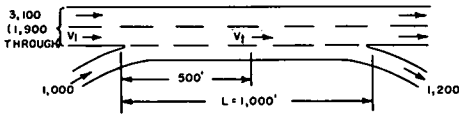
factor of 0.91, and the following traffic pat-
tern: Upstream freeway volume = 3,100 vph,
of which 1,900 are through vehicles, on-
ramp volume = 1,000 vph, off-ramp volume
= 1,200 vph, and no on-ramp to off-ramp
traffic. Geometrics are ideal.

Determine whether or not this pair of
junctions meets level D requirements.

Solution:

On this relatively short section the center,
or 500-ft, point appears to be the critical
point. Table 8.3 indicates that at this point
lane 1 will be handling 25 percent of the
through traffic, or $0.25 \times 1,900 = 475$ vph.
Case II(a) of Figure 8.23 indicates that
80 percent of the on-ramp traffic ($0.80 \times 1,000$), or 800 vph, will be in lane 1, as will
24 percent of the off-ramp traffic ($0.24 \times 1,200$), or 288 vph, for a total of $475 + 800 + 288 = 1,563$ vph in lane 1. Using Ta-
ble 8.1, this is found to be satisfactory when
compared to the merge control value of
1,650 vph for level D, PHF = 0.91.

The weaving that takes place in a 500-ft
section must also be determined. In the
same example, it can be seen that in the
first 500 ft 80 percent of the on-ramp traffic
will weave with 76 percent of the off-ramp
traffic. This would be $0.80 \times 1,000 + 0.76 \times 1,200 =$ approx. 1,700 vph. Inasmuch as this
is slightly more than the allowable weave
given in Table 8.1 of 1,650 vph per 500 ft
of roadway for level D, some turbulence can

be expected. This example illustrates the
undesirability of having noses of high-vol-
ume successive on- and off-ramps as close
together as 1,000 ft, even with an auxiliary
lane.

Conclusion:

Level D requirements are not fully, but
very nearly, met.

EXAMPLE 8.7 (Showing use of Fig. 8.24)

(Note: Figure 8.24 incorporates a sum-
mary of this simple example of the basic
procedure involved in use of Figure 8.24.)
Problem:

The given design is a 4-lane freeway (two
lanes in each direction) with a 1-lane on-
ramp entering (see sketch). The upstream
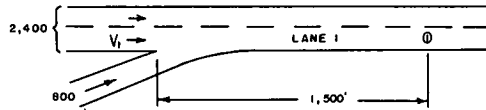
freeway volume is 2,400 vph and the on-
ramp volume is 800 vph. Geometrics are
ideal.

Determine volume of on-ramp traffic still
in lane 1 at point ①, 1,500 ft downstream of
the junction.

Solution:

(a) Basic calculation.
From Table 8.3, for the given
through volume of 2,400 vph, 30 per-
cent is likely to stay in lane 1.
$2,400 \times 0.30 = 720$ vph.
At point ①, from Figure 8.23a for
distance = 1,500 ft, 30 percent of the
on-ramp traffic will be in lane 1.
(Caution: It is found that Note 2 on
the figure must be considered before
the problem is complete, as the in-
tercept is below the dashed line.)
$800 \times 0.30 = 240$ vph.
$720 + 240 = 960$ vph like to be in
lane 1 at ①.

(b) Check calculation, applying Note 2.
Consider ramp traffic as through
traffic.

Through traffic = 2,400 + 800 = 3,200 vph.
From Table 8.3, for 3,200 vph, 40 percent of the traffic will stay in lane 1.

  $3,200 \times 0.40 = 1,280$ vph likely to be in lane 1 at ①.

(c) Conclusion:
Procedure (b) gives greater result than (a) (1,280 vs 960), so use 1,280 vph.

EXAMPLE 8.8

Problem:

Given: 8-lane freeway with 4 lanes one way and no auxiliary lane.
Normal percentage trucks, no excessive grades or curvature.
On- and off-ramps, 2,000 ft between noses; no other ramps within 4,000 ft.
Peak-hour factor = 0.83.
Traffic: A-B = 4,200 vph; A-Y = 500 vph; X-B = 1,200 vph; X-Y = 0 vph; total = 5,900 vph between ramps.
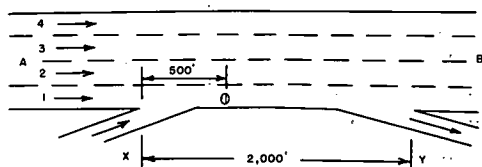
Determine whether or not the design satisfies the requirements for level of service D at point ①.

Solution:

(a) Volume across through freeway lanes = 5,900 vph.
5,900 < 6,000 vph, from Table 8.1; satisfactory.

(b) Lane 1 volume at ①.
Through traffic in right lane (Table 8.3) = $0.08 \times 4,200 = 336$.
On-ramp traffic in right lane (Fig. 8.23a) = $1.00 \times 1,200 = 1,200$.
Off-ramp traffic in right lane (Fig. 8.23a) = $0.79 \times 500 = 395$.
Total = 1,931 > 1,500, from Table 8.1; unsatisfactory.

(c) Conclusion:
Overloaded for level D, as the geometric and traffic pattern is not satisfactory. (See proposed redesigns, Examples 8.9 and 8.10).

EXAMPLE 8.9

Problem:

Given: Same conditions as Example 8.8, except that an additional upstream on-ramp is added, as shown.
Traffic: A-B = 4,200 vph; A-Y = 500 vph; X_1-B = 500 vph; X_2-B = 700 vph; X_1-Y = 0 vph; X_2-Y = 0 vph; total = 5,900 vph between on- and off-ramps.
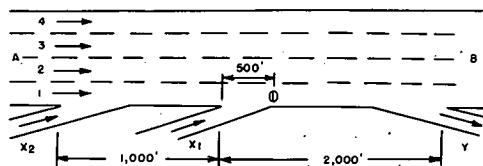
Determine whether or not this redesign satisfies the requirements for level of service D at point ①.

Solution:

(a) Volume across through freeway lanes = 5,900 vph.
5,900 < 6,000, from Table 8.1; satisfactory.

(b) Lane 1 volume at ①.
Traffic in right lane from upstream of ramp X_2 (test two alternate methods; use the larger result).
Alternate 1:
Through traffic in right lane (Table 8.3) = $0.08 \times 4,200 = 336$
X_2 on-ramp traffic in right lane (Fig. 8.23a) = $0.30 \times 700 = 210$
Total = 546 (use).
Alternate 2:
Consider X_2 on-ramp traffic as through (Table 8.3) = $0.09 \times 4,900 = 440$ (reject).
X_1 on-ramp traffic in right lane (Fig. 8.23a) = $1.00 \times 500 = 500$.
Off-ramp Y traffic in right lane (Fig. 8.23a) = $0.79 \times 500 = 395$.
Total = 1,441 < 1,500, from Table 8.1; satisfactory.

(c) Weaving in 500 ft (Fig. 8.23a).
On-ramp $X_1 = (1.00 - 1.00)(500)$
$= 0.$
On-ramp $X_2 = (0.60 - 0.30)(700)$
$= 210.$
Off-ramp $= (0.79 - 0.63)(500)$
$= 80.$
Total weave $= 270 < 1,500,$ from Table 8.1; satisfactory.

(d) Conclusion: Meets level D requirements; satisfactory. Thus, the unsatisfactory condition in Example 8.8, which has a single high-volume on-ramp 2,000 ft from a 500-vph off-ramp, can be made acceptable if conditions are such that the on-ramp can be split into two on-ramps so that some X_2 on-ramp traffic can distribute into available gaps in the left lanes before the X_1 on-ramp traffic enters.

EXAMPLE 8.10

Problem:

Given: Same conditions as Example 8.8, except that an auxiliary lane is provided.
Traffic: A-B $= 4,200$ vph; A-Y $= 500$ vph; X-B $= 1,200$ vph; X-Y $= 0$ vph; total $= 5,900$ vph between ramps.
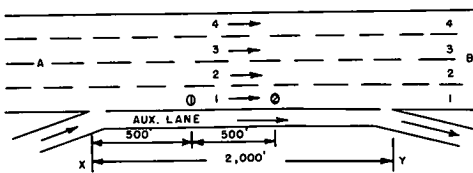
Determine whether or not the design satisfies the requirements for level D at points ① and ②.

Solution:

(1) Point ① (500 ft downstream of on-ramp):
   (a) Lane 1:
      Through traffic (Table 8.3) $= 0.08 \times 4,200 = 336.$
      On-ramp traffic (Fig. 8.23b) $= 0.50 \times 1,200 = 600.$

Off-ramp traffic (Fig. 8.23b) $= 0.29 \times 500 = 145.$
Total, lane $1 = 1,081 < 1,500,$ from Table 8.1; satisfactory.

(b) Auxiliary lane:
   On-ramp traffic $= 0.50 \times 1,200 = 600.$
   Off-ramp traffic $= 0.50 \times 500 = 250.$
   Total, auxiliary lane $= 850 < 1,500,$ from Table 2.1; satisfactory.

(c) Volume across all freeway lanes:
   5,900 (Total volume) $-$ 850 (Volume in auxiliary lane) $= 5,050.$
   $5,050 < 6,000,$ from Table 8.1; satisfactory.

(d) Weaving volume in 500-ft segment:
   From Fig. 8.23b, On-ramp $= 0.50 \times 1,200 = 600.$
   Off-ramp $= 0.50 \times 500 = 250.$
   Total weave $= 850 < 1,500,$ from Table 8.1; satisfactory.

(e) Conclusion: Conditions for level D are met; the design is satisfactory at ①.

(2) Point ② (1,000 ft downstream of on-ramp):
   (a) Lane 1:
      Through traffic (Table 8.3) $= 336.$
      On-ramp traffic (Fig. 8.23b) $= 0.66 \times 1,200 = 792.$
      Off-ramp traffic (Fig. 8.23b) $= 0.19 \times 500 = 95.$
      Total, lane $1 = 1,223 < 1,500,$ from Table 8.1; satisfactory.

   (b) Auxiliary lane:
      On-ramp traffic $= 0.14 \times 1,200 = 168.$
      Off-ramp traffic $= 0.76 \times 500 = 380.$
      Total, auxiliary lane $= 548 < 1,500,$ from Table 8.1; satisfactory.

   (c) Volume across all freeway lanes:
      $5,900 - 548$ (Volume in auxiliary lane) $= 5,352.$
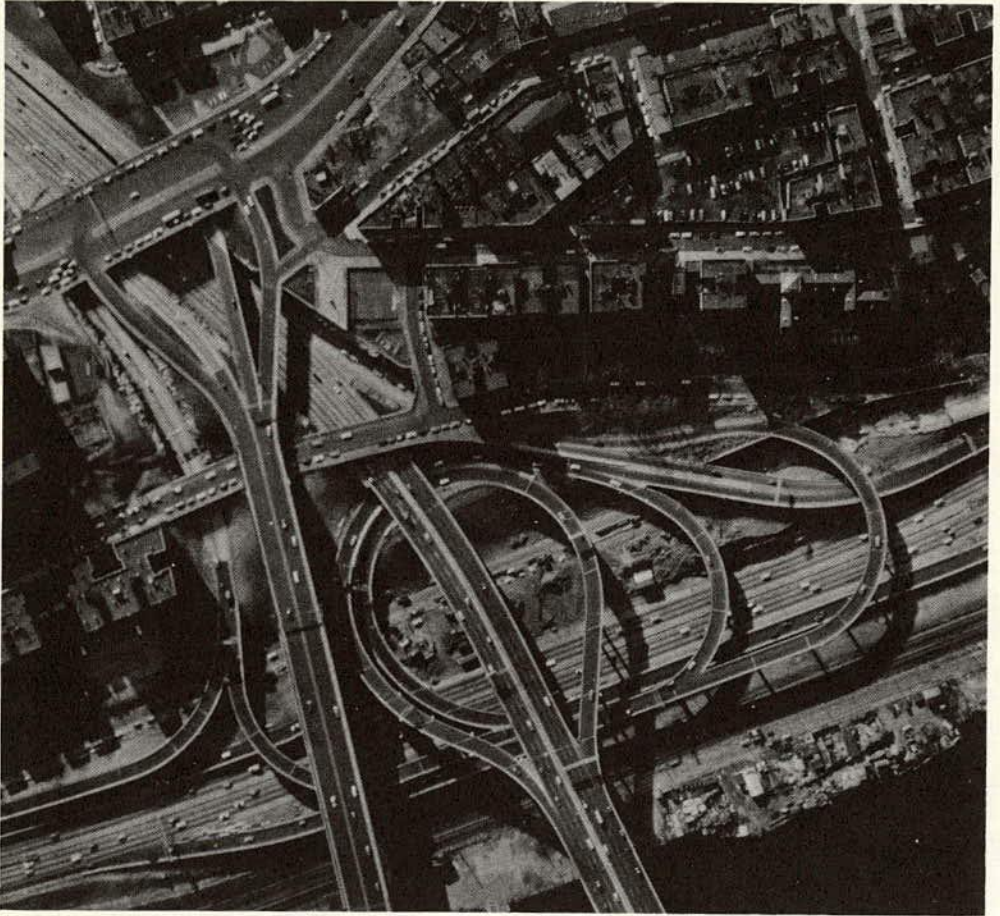
Ramp complex in high-density urban area.

5,352 < 6,000, from Table 8.1; satisfactory.

(d) Maximum weave occurs in 1st 500 ft; further check is unnecessary, since it was satisfactory.

(e) Conclusion: Conditions for level D are met; the design is satisfactory at ②.

(3) Overall conclusion: The unsatisfactory condition in Example 8.8 can thus be made acceptable by adding an auxiliary lane between the on- and the off-ramp.

COMPUTATIONAL PROCEDURES — LEVEL E (CAPACITY)

Level of service E, or capacity, involves still greater volumes, at a poorer quality of service, than the level D operation on which the foregoing procedures have been based. As discussed earlier in this chapter, many ramp-freeway merges and diverges have consistently shown a maximum volume that has reasonable possibility of occurring over a full hour of 2,000 to 2,100 vph. This occurs with sufficient frequency to justify the listing
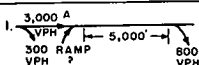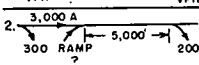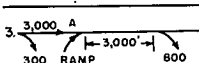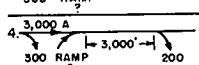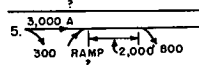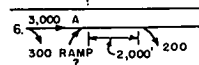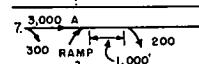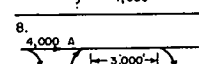
of 2,000 vph as merge and diverge capacity in Table 8.1.

Nevertheless, the specific conditions of geometric design and traffic characteristics which permit the consistent attainment of 2,100 vph are not well identified as yet. It is neither feasible nor wise, therefore, to present procedures to predict operation at this level. An estimate can be made by substituting a 2,000-vph rate for the 1,800-vph rate assumed as a maximum rate of flow for service level D, but operation with volume rates of 2,000 vph per lane is likely to be unreliable. If there are any unusual conditions, the 2,000-vph rate may not be attained.

Unstable flow exists at level of service E, with "breakdown" likely. Further demand increase will exceed the capacity of the merging or diverging area. The result will be forced flow operation, level of service F. Long queues may develop, with accompanying delays to motorists. At most locations, particularly those where drivers have long experience in congested freeway driving,

TABLE 8.4—EXAMPLES OF SERVICE VOLUMES, LEVELS C AND D
(PEAK-HOUR FACTOR = 0.83)

GEOMETRIC AND TRAFFIC PATTERN[a]	ESTIMATED NUMBER OF VEHICLES IN LANE 1 AT (A)[b] (vph)		ALLOWABLE RAMP VOLUME (vph)		ALLOWABLE DOWNSTREAM FREEWAY VOLUME (vph)	
	SERVICE LEVEL C, MERGE 1,400 vph	SERVICE LEVEL D, MERGE 1,500 vph	SERVICE LEVEL C, MERGE 1,400 vph	SERVICE LEVEL D, MERGE 1,500 vph	SERVICE LEVEL C, MERGE 1,400 vph	SERVICE LEVEL D, MERGE 1,500 vph
1. 3,000 A / 5,000' / 300 RAMP 800 VPH ?	690	180	710	1,320	3,710	4,320
2. 3,000 A / 5,000' / 300 RAMP 200 ?	610	180	790	1,320	3,790	4,320
3. 3,000 A / 3,000' / 300 RAMP 800 ?	755	500	645	1,000	3,645	4,000
4. 3,000 A / 3,000' / 300 RAMP 200 ?	625	260	775	1,240	3,775	4,240
5. 3,000 A / 300 RAMP 2,000 800 ?	840	760	560	740	3,560	3,740
6. 3,000 A / 300 RAMP 2,000' 200 ?	650	320	750	1,180	3,750	4,180
7. 3,000 A / 300 RAMP 1,000' 200 ?	715	370	685	1,130	3,685	4,130
8. 4,000 A / 3,000' / 300 RAMP 800 ?	1,000	560	0[c]	500[d]	4,000[c]	4,500[d]

[a] All examples are three lanes in one direction, without auxiliary lane.
[b] Excluding on-ramp vehicles.
[c] Maximum total freeway volume is 4,000, so maximum ramp volume is 0 vph.
[d] Maximum total freeway volume is 4,500, so maximum ramp volume is 500 vph.

only part of the excess will normally be deducted from the on-ramp volumes; the remainder of the loss will be in the freeway flow. Suppose, for example, that an on-ramp demand is 1,200 vph, but that the capacity as calculated in this section is only 600 vph. This does not mean that only 600 vph will get on the freeway at this location. With a demand of 1,200 vph, the difference of 600 vph will be partly waiting in a queue on the ramp, and partly in a queue on the freeway. The freeway flow will have "broken down" with long irregular queuing, mostly in the right lane but with spill-over queuing and stop-and-go operation in adjacent lanes. This type of operation results in hazardous lane-changing upstream. Further, the ramp volume under "stop-and-go" operation, which is level of service F, will be limited, in many cases, to a maximum of about 900 vph. This is because, as discussed earlier, lane 1 vehicles will alternate with ramp vehicles entering the merging area, which has a capacity of approximately 1,800 vph under these conditions. Actual volumes carried at any specific location may well be considerably less, depending on other local conditions.

Comparison of Level C and Level D Calculations

In order to aid the reader in comparing the essential differences between the nomographic solution for level of service C or better, as outlined earlier in this chapter, and the chart solution for level D just outlined, Table 8.4 has been prepared to show examples of variations in results.

It should be noted that the change in service volume D of the on-ramp is greater than the change in service volume C as the distance between the on-ramp and the next downstream off-ramp decreases, or as the volume at the off-ramp decreases. By comparing cases 5 and 6 in Table 8.4, it is seen that service volume C of the on-ramp is increased only 190 vph when the off-ramp volume is decreased by 600 vph (from 800

to 200), whereas service volume D of the on-ramp is increased by 440 vph. By comparing cases 1 and 5, it is seen that service volume C of the on-ramp is increased by only 150 vph when the distance to a downstream off-ramp is increased from 2,000 ft to 5,000 ft, whereas service volume D is increased by 580 vph.

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FREEWAYS AND OTHER EXPRESSWAYS

Freeways and other expressways are intended to provide a generally high level of service to their users and to the communities which they serve, offering rapid traffic movement without outside interference. They accomplish this by eliminating direct service to abutting properties in favor of exclusive service to moving traffic. This results in high user demand for these highways. Consequently, in some cases, particularly in urban areas where freeway networks remain incomplete, soon after their completion these highways have experienced peak-period traffic demands which equal or exceed their capacities. This early congestion, although evidence that at least certain components of these highways are not providing their intended level of service during peak periods, does not detract from the value of the high level of service provided during the remaining periods of the day when as much as 80 percent of the daily traffic is served.

It is important to understand both the fundamental operational characteristics of basic sections of freeways and expressways, unencumbered by entrance or exit points and other outside influences, and the influence of elements such as ramp junctions, weaving sections, and other restrictions on this operation. The primary purpose of this chapter is to present basic procedures for determination of service volumes and capacities of basic sections of freeways and expressways. However, considerable emphasis is also placed on steps which will minimize the possibility of "spot" or temporary overloading at any point, whatever the cause may be. In direct use, given the traffic demand, this engineering approach involves selection of the desired level of service, followed by design of all portions and features of the highway in consonance with that level.

If one element of a freeway functions at a lower level than the selected level of service, it may limit the level of service over a substantial portion of the freeway section; therefore, every element must be in proper balance with all other elements, with due regard for the traffic variations along the section caused by entering and leaving traffic. This balance does not necessarily imply identical operating speeds or conditions throughout. Drivers will accept somewhat lower speeds through critical sections such as steep grades, weaving areas, and ramp junctions, as well as through intersections on expressways, than they will elsewhere, for any given level of service.

This chapter furnishes, either directly or through reference to other chapters, information and procedures sufficient to permit evaluation of the capacities and levels of service of complete sections of freeways and other expressways, both rural and urban, involving not only the through lanes, but also critical areas of operation, including upgrades, weaving areas, ramp entrances and exits, and, in the case of other expressways, intersections at grade.

Only multilane freeways are considered in this chapter. Two-lane highways with full control of access are handled by the methods described in Chapter Ten.

BASIC LEVELS OF SERVICE

The objective of modern freeways and other expressways is to provide good service for high volumes of traffic. Because freeways are high-type highways, many have, or approach, ideal geometrics. That is, freeways come the closest of any highway type to duplicating the "ideal" geometric conditions for vehicular operation defined in Chapter Four, including 12-ft lanes, adequate lateral clearances and shoulders, and alinement for 70-mph average highway speed. The "ideal"

traffic condition of no trucks is seldom attained, however. The controlled-access features and one-way roadways provided by these divided highways reduce potential restrictions, conflicts and hazards to traffic flow from external influences, permit higher levels of service for given traffic volumes, and usually allow greater capacity per lane.

The capacity under "ideal" conditions for average multilane facilities is given in Chapter Four as 2,000 passenger cars per lane per hour, average for all lanes at about 30 mph. For freeways this capacity is generally attained at the somewhat higher speed of 35 mph. In fact, on a very few freeways an optimum average volume per lane of 2,100 pcph at 40 mph has been attained on occasion. Because it can be attained only under very special circumstances, this average is not considered a "reasonably attainable" volume, in terms of the definition of capacity contained in Chapter Two.

High-type parkways with freeway geometrics in level terrain may be entirely ideal, for all practical purposes, because they carry no trucks; thus, traffic conditions as well as geometric conditions are "ideal." Hence, it is entirely possible for a parkway to have a capacity of 2,000 vehicles (all passenger cars) per lane per hour. Similarly, a modern freeway may well have close to that capacity, the only downward adjustments being for the trucks present in the flow, and for grades, if present. Thus, the typical freeway speed distributions and speed-volume relationships for ideal conditions which were shown in Chapter Three (as Fig. 3.23 and as Figs. 3.35, 3.38, and 3.41, respectively) may represent actual operation on a substantial number of freeways.

Nevertheless, many older and lower-standard freeways and parkways exist and continue to provide good service. On these, account must be taken of several of the remaining factors discussed in Chapter Five, such as lane width and lateral clearance, presence of shoulders, and average highway speed. In some cases the effect varies, depending on the level of service involved. Due to the lower design standards, a few of these highways may be incapable of providing service at level A, and in extreme cases level B may be unattainable. The previously mentioned

curves in Chapter Three do not correctly represent such highways. Therefore, they should not be used for computational purposes.

Level of service, by definition based on practical necessity, must apply to a section of roadway. It reflects the average operating conditions in the section under consideration. In Chapter Four, the factors considered in determining levels of service were set forth, and it was concluded that two factors—a measure of travel speed and the ratio of demand or service volume to capacity—were most feasible for use in identifying these levels. In this chapter, therefore, operating speeds, as related to ratios between demand volumes or service volumes and capacity (v/c ratios), are used as the determinants of level of service.

Operating speed has been defined as the maximum safe speed for given traffic conditions that an individual vehicle can travel if the driver so desires, without exceeding the design speed at any point. It represents a basically theoretical speed unlikely to be identified in actual traffic, although undoubtedly occasionally approximated by a few of the faster, though not reckless, drivers. A given freeway or expressway will have a free-flow operating speed, or a maximum safe speed at extremely low volumes, governed by the physical characteristics of the roadway. As volumes increase, operating speeds will drop, through the complete range of levels of service up to capacity. An expressway's operating speed will, in addition, be influenced by infrequent traffic interruptions, such as high-type at-grade intersections. A series of operating speed values define the limits of the several levels of service from a speed standpoint.

The v/c ratio values used to define the second fundamental scale of level of service limits in the descriptions that follow are based on ideal alinement, 70-mph average highway speeds, and two lanes in one direction. The basic ratios thus established are independent limits, developed from a volume standpoint alone. In practice, however, their application to problems involving highways of lower design standards is unrealistic and of little use, because they usually would represent service volume levels considerably
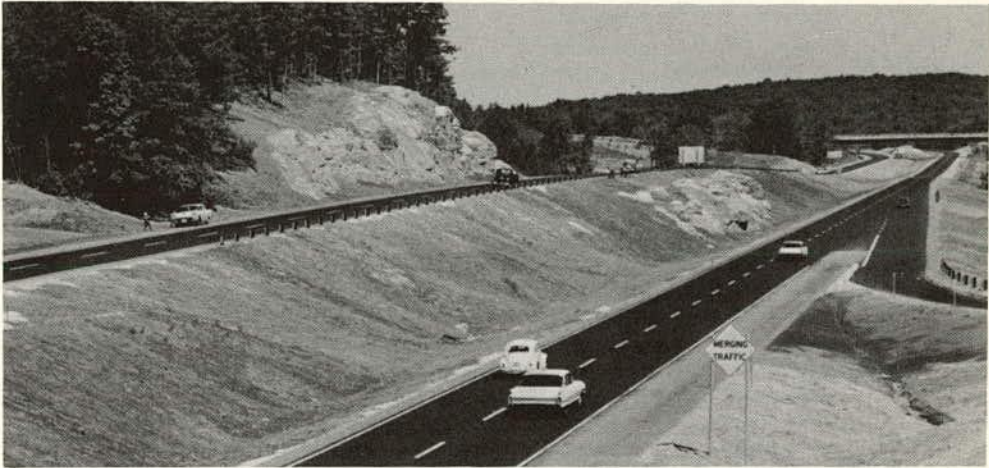
Rural freeway in rolling terrain, showing independent roadway design.

higher than could be attained while the operating speed limit for the same level of service was being met. Hence, in the procedures that follow, approximate working *v/c* limits for lower average highway speeds are also given. Somewhat larger basic *v/c* ratio limits apply in many cases where more than two lanes exist in a given direction.

The traffic volume seldom remains constant over any appreciable distance. Instead, it changes at entrance and exit points. Therefore, the ratio of demand volume to any selected service volume or to capacity will vary along the expressway. Each roadway section and each critical capacity location must be examined in relation to the selected level of service and the design developed accordingly, so that operating conditions will be balanced. If the demand exceeds the capacity at any location, this will be a critical point, and the level of service may be adversely influenced for a long distance upstream. Where demand exceeds only some designated service volume lower than capacity, on the other hand, the area affected upstream and downstream of a single restriction may be very small.

The same criteria apply to all freeways, whether rural or urban, but differing levels of service may well be chosen for design purposes in the two cases. That is, there are no basic differences between rural and urban freeway capacity and level of service determination procedures, but only differences in the applications made of them.

Operating characteristics at the several levels of service are next described.

Level of Service A

Free-flow operation is defined as that flow condition in which a vehicle essentially is not affected by other vehicles in the traffic stream, and selection of speed is based on the individual driver's choice and on roadway design features.

Level of service A is defined as free-flow operation, with operating speeds at or greater than 60 mph. This is equivalent to a requirement that operating speeds be not more than 10 mph below those possible with ideal geometrics under very low volume conditions. The service volume at this level is 1,400 passenger cars per hour total for two lanes in one direction under ideal conditions, (or an average of 700 passenger cars per lane per hour). Free flow may occur even on expressways with relatively poor alinement, provided the volume is sufficiently low. However, such operation necessarily occurs at lower speeds. Where free-flow operating speeds for through traffic fall below 60 mph, the quality of service does not

meet the requirements for level A; consequently, that level will never be attained on the particular highway involved. Average speeds are most likely to be affected by speed limits at level A.

On four-lane freeways with two lanes in each direction, it has been found that speed adjustments made necessary by other traffic, rather than by choice, become significant at volumes of approximately 35 percent, or one third, of capacity. The faster group of drivers begins to be reluctant to use the right lane for fear of being "trapped" in that lane behind a slow vehicle while a platoon of fast vehicles develops and passes the slow vehicle.

Curves showing speed versus traffic volume may not be sensitive enough to pinpoint this effect. The fast platoons in the left lane may still be traveling at speeds near the desired operating speeds, but the inherently slower drivers, as well as those that are "trapped" in the right lane with them, will be traveling somewhat slower. At these volumes of about 35 percent of capacity there may be long intervals when only isolated single vehicles are passing, all such vehicles being "free-moving," while at other times platoons with accompanying short headways may exist in both lanes. About one-half of the vehicles will still be under free-flow conditions, but the remainder of the drivers will be influenced by the presence of other traffic. This type of operation represents the transition between free and stable flow, and defines the dividing line between level of service A and level of service B, which is described as the limit of level of service A.

Where, as on 6-lane and 8-lane freeways, there are three or more lanes in one direction, the influence of slow vehicles on the traffic stream as a whole is diminished. The probability of slower vehicles obstructing the traffic stream traveling abreast is greatly reduced and freedom to maneuver and pass is greatly increased. At level A, therefore, with three or more lanes in one direction, each additional lane provided above two will result in a one-way service volume increase of about 1,000 passenger cars per hour, which is approximately 1.5 times the average volume per lane of two lanes in one direction. This increase in efficiency is reflected in

somewhat larger percentages of capacity attained, as compared to the 35 percent for the two lanes in one direction.

Level of Service B

Level of service B is in the higher speed range of stable flow. For freeways and expressways, it is defined by the requirements that operating speeds be at or greater than 55 mph and that the service volume on two lanes in one direction not exceed 50 percent of capacity. This gives a maximum service volume of 2,000 passenger cars per hour total for two lanes in one direction under ideal conditions (or an average of 1,000 passengers cars per lane per hour). If an operating speed of 55 mph for through traffic cannot be maintained for this service volume, the quality of service does not meet the requirement for level B.

At this volume level of 50 percent of capacity, the possibility of free-flow operation has been further reduced. There continue to be significant speed differences between lanes, but the highest operating speed a driver can maintain is now in the range of 75 to 90 percent of that attainable under free flow. Speed has now become primarily a function of traffic densities. This defines the dividing line between level of service B and level of service C, which is described as the limit of level B.

Again, as was true for level A, each additional lane above two in one direction provides about 1.5 times the average service volume per lane of two lanes, here about 1,500 passenger cars per lane per hour.

Level of Service C

Further increases in demand volume are accompanied by a resultant decrease in operating speeds, into level of service C. Operation at this level, although still in the range of stable flow, is critical enough so that, unlike levels A and B, rates of flow within a period shorter than an hour must be considered. For freeways, a 5-min short period has been adopted as the standard. In general, the requirements for level of service C are an operating speed of at least 50 mph and a service flow rate on two lanes in one direction not exceeding 75 percent of the capacity rate, with service volumes de-

veloped from these rates through application of the appropriate peak-hour factor. Further, under ideal conditions for two lanes in one direction, the peak 5-min flow rate cannot exceed 3,000 passenger cars per hour total for one direction (an average of 1,500 passenger cars per lane per hour).

Variations in uninterrupted traffic flow within the peak hour have been discussed in Chapters Three, Five, and Eight. It has been indicated that the rate of flow for the highest 5-min interval of an hour is always higher than the rate of flow for the whole hour. This is due to a natural statistical variability among the twelve 5-min intervals, as well as to a variation in demand. For freeways and expressways the peak-hour factor has been described as the ratio of the whole-hour volume to the highest rate of flow occurring during a 5-min interval within the peak hour (that is, to 12 times the actual 5-min flow).

In large metropolitan areas of over a million population, the rate of flow for a whole hour will be about 0.91 of the peak 5-min rate of flow within the hour. For areas between 500,000 and 1,000,000 population, a peak-hour factor of 0.83 is suggested, while in areas under 500,000 population, a peak-hour factor of 0.77 has been found to be satisfactory.

It follows that if the level C peak-flow rate of 3,000 vph total in one direction on a 4-lane freeway is not to be exceeded in any 5-min peak period within the peak hour in a large metropolitan area (PHF=0.91), the volume in one direction cannot exceed 2,750 passenger cars in a whole hour (3,000 ×0.91); in a smaller area (PHF=0.77) that volume cannot exceed 2,300 passenger cars in one direction in a whole hour (3,000 ×0.77). At both locations the probability of failure to meet level C flow limits will be about the same if the computed volumes are exceeded. When volumes reach about 75 percent of capacity on freeways and other expressways, with due allowance for peaking, operating speeds are about two-thirds of those attainable during free-flow conditions. The differences in speed and volume between lanes are still significant, in the order of 5 mph and up to several hundred vehicles per hour. A driver's desire to maximize operating speed may require almost con-

Major elevated urban freeway network.

tinuous use of the left lane, where he will tolerate average headways of approximately 2 sec. Traffic is now approaching the maximum volume that can be maintained for extended periods of time, with continuing capability for recovery from momentary conflicts and obstructions without undue delay. This condition defines the dividing line between level of service C and level of service D.

With only two lanes available in one direction, the slower drivers generally will continue to use the right lane. However, given three or more lanes, even some of the slower

drivers may now be hesitant about using the right-hand lane, fearing possible conflicts with entering and leaving traffic. They may move into adjoining lanes to avoid such conflicts. At level C, therefore, on highways with more than two lanes in one direction, the increase in efficiency provided by the additional lanes is somewhat reduced as compared to levels A and B; each additional lane will provide a one-way peak flow rate increase of approximately 1.2 times the average peak flow rate per lane of two lanes in one direction, or 1,800 passenger cars per hour.

Level of Service D

For level of service D, as for level of service C, any discussion of freeway volume must be qualified by consideration of the peak-hour factor.

In level D, which is in the lower speed range of stable flow with volumes higher than in level C, traffic operation approaches instability and becomes very susceptible to changing operating conditions. Operating speeds generally are in the neighborhood of 40 mph, and service flow rates do not exceed 0.90 of capacity rates (with service volumes again obtained through application of the appropriate peak-hour factor). Under ideal conditions on a four-lane freeway the peak 5-min flow rate cannot exceed 3,600 passenger cars per hour, total for one direction (an average of 1,800 passenger cars per lane per hour).

Except in those cases where a fully balanced design has been attained, potential conflict points begin to have a much greater effect on operations. Traffic may operate near capacity at these points, although at least partial freedom of movement may well remain between them. These conflict points, or potential bottlenecks, begin to meter the flow throughout the entire roadway section. Therefore, new designs normally would not be based on this level.

A basic fact about traffic flow appears to be that when average headways of less than 2 sec occur over sustained periods, the momentary fluctuations or obstructions which almost certainly will occur will have a detrimental effect on operating conditions. This is equivalent to an average flow rate of 1,800 vehicles per hour per lane across all freeway lanes (90 percent of the overall one-directional capacity), regardless of the number of lanes; additional lanes above two no longer improve average efficiency per lane. Traffic densities in all lanes are fairly uniform, regardless of the number of lanes, with the somewhat higher speeds in the left lanes providing higher lane service volumes. This represents a tolerable limitation on the uniform functioning of the expressway throughout the time period. These limits define the division between level of service D and level of service E, or the dividing point between stable and unstable flow.

Level of Service E

Level of service E is the area of unstable flow, involving overall operating speeds of about 30-35 mph, and involving volumes approaching and at capacity, or about 2,000 passenger cars per lane per hour under ideal conditions. Service volume is almost strictly regulated by the capacity at critical locations, with traffic being metered through each restriction, but demand does not greatly exceed capacity, so long backups do not develop upstream. Operating conditions may involve either fairly uniform speeds of about 50 percent of free-flow operating speed through the entire section, or a more intermittent type of operation upstream from a constriction where storage is taking place; such constrictions may be either permanent (inherent in the geometrics), or temporary, such as minor accidents or broken-down vehicles.

Traffic flow within the hour will, therefore, show relatively little fluctuation, inasmuch as traffic is in effect being successively metered along the highway; but still there will be variations. Until it becomes extreme this fluctuating traffic movement along the highway can be accommodated, but the public considers this to be very poor service; as actual stoppages become more frequent their effect tends to be cumulative, increasingly detrimental, and finally constant, with traffic operations reverting to forced-flow conditions. This marks the division between level of service E and level of service F.

Although level E operation is unstable, it is found on many freeways under peak-

A complex urban interchange between two freeways.

period conditions, particularly where demand increases gradually. Design at this level should never be attempted, however.

Level of Service F

This level describes a forced-flow condition in which the expressway acts as storage for vehicles backing up from a downstream bottleneck. Operating speeds range downward from those at capacity (at or near 30 mph) to those during stop-and-go type operation, and can drop to zero in the extreme case of a complete jam. Volumes vary widely, depending principally on downstream capacity. This service is unacceptable.

Very often, where a sudden demand surge occurs, operation may by-pass level E completely, passing directly (that is "breaking down") from level D into this forced flow level F.

Descriptions of the several levels of service on freeways and expressways of various numbers of lanes, including limiting operating speeds and v/c ratios, both for ideal and restricted alinement, together with corresponding actual maximum service volumes,

peak flow rates, and capacities for various numbers of lanes under ideal uninterrupted flow conditions, are summarized in Table 9.1.

CRITICAL ELEMENTS REQUIRING CONSIDERATION

The previous section has analyzed levels of traffic service on freeways and expressways primarily from the standpoint of a basic through roadway section, level, with excellent geometrics and alinement, free of entrance and exit ramps and other special features, and carrying passenger cars only. It presents an overall evaluation of the service provided, making only brief mention of the need for harmony among the elements making up the section.

Actual roadway and traffic conditions at various points along the highway will not be constant, although on most freeways relatively high geometric standards can be expected throughout. During moderate to heavy volumes, operating conditions vary along the roadway, fluctuating due to changes in physical roadway conditions,

TABLE 9.1—LEVELS OF SERVICE AND MAXIMUM SERVICE VOLUMES FOR

LEVEL OF SERVICE	TRAFFIC FLOW CONDITIONS		SERVICE VOLUME/CAPACITY (v/c) RATIO[a]				
	DESCRIPTION	OPERATING SPEED[a] (MPH)	BASIC LIMITING VALUE FOR AVERAGE HIGHWAY SPEED (AHS) OF 70 MPH, FOR:			APPROXIMATE WORKING VALUE FOR ANY NUMBER OF LANES FOR RESTRICTED AVERAGE HIGHWAY SPEED OF	
			4-LANE FREEWAY (2 LANES/ DIRECTION)	6-LANE FREEWAY (3 LANES/ DIRECTION)	8-LANE FREEWAY (4 LANES/ DIRECTION)	60 MPH	50 MPH
A	Free flow	≥60	≤0.35	≤0.40	≤0.43	—b	—b
B	Stable flow (upper speed range)	≥55	≤0.50	≤0.58	≤0.63	≤0.25	—b
Peak-Hour Factor (PHF)c							
C	Stable flow	≥50	≤0.75 (PHF)	≤0.80 (PHF)	≤0.83 (PHF)	≤0.45 (PHF)	—b
D	Approaching unstable flow	≥40	≤0.90 (PHF)			≤0.80 (PHF)	≤0.45 (PHF)
Ef	Unstable flow	30–35 e	≤1.00				
F	Forced flow	<30 e	←———— Not meaningful ————→				

a Operating speed and basic v/c ratio are independent measures of level of service; both limits must be satisfied in any determination of level.
b Operating speed required for this level is not attainable even at low volumes.
c Peak-hour factor for freeways is the ratio of the whole-hour volume to the highest rate of flow occurring during a 5-min interval within the peak hour.
d A peak-hour factor of 1.00 is seldom attained; the values listed here should be considered as maximum average flow rates likely to be obtained during the peak 5-min interval within the peak hour.
e Approximately.
f Capacity.

variations in demand volume, intervehicular conflicts at weaving and merging areas, and the influences of traffic control features. The operating characteristics at each such location must be investigated for the effect on roadway capacity and level of service. Unless the adopted level of service is met as a minimum at every point on a highway, the restrictions will cause traffic operations to fall below the desired level of service, often for extended distances. In the extreme, where operation at capacity is required, any restrictions will be capacity limitations (bottlenecks) preventing full use of the remainder of the section. An approach to freeway level of service thus requires analysis of each potential restriction or bottleneck within the roadway section.

FREEWAYS AND EXPRESSWAYS UNDER UNINTERRUPTED FLOW CONDITIONS

MAXIMUM SERVICE VOLUME UNDER IDEAL CONDITIONS, INCLUDING 70-MPH AVERAGE HIGHWAY SPEED (TOTAL PASSENGER CARS PER HOUR, ONE DIRECTION)

4-LANE FREEWAY (2 LANES ONE DIRECTION)				6-LANE FREEWAY (3 LANES ONE DIRECTION)				8-LANE FREEWAY (4 LANES ONE DIRECTION)				EACH ADDITIONAL LANE ABOVE FOUR IN ONE DIRECTION			
1400				2400				3400				1000			
2000				3500				5000				1500			
0.77	0.83	0.91	1.00d	0.77	0.83	0.91	1.00d	0.77	0.83	0.91	1.00d	0.77	0.83	0.91	1.00d
2300	2500	2750	3000	3700	4000	4350	4800	5100	5500	6000	6600	1400	1500	1650	1800
2800	3000	3300	3600	4150	4500	4900	5400	5600	6000	6600	7200	1400	1500	1650	1800
4000e				6000e				8000e				2000e			

←————————— Widely variable (0 to capacity) —————————→

The traffic operating characteristics at, and therefore the design of, each critical section should be in harmony with the level of service adopted as a minimum for the roadway as a whole. Ideally, harmony would require total uniformity in level of service at all points, but this is not always feasible in practice. Instead, an average level of service throughout the roadway section must be created such that at each critical location operating levels will at least equal the minimum level of service adopted. This means that portions of the section will have a level of service somewhat higher than the restricted points, although not necessarily sufficiently higher to fall in a different level of service classification. Never should the

Restricted geometrics, including substandard lateral clearances, adversely influence operation on this section of an older urban freeway.

variation between adjoining subsections exceed one level.

The following situations are among the common ones which require testing and analysis: Sudden increases in traffic demand (at on-ramps or weaving areas); creation of intervehicular conflicts within the traffic stream by changing roadway conditions (at points of reduction in number of traffic lanes, off-ramps, grades, weaving areas); variation in nature of traffic demand (varying percentages of trucks); adverse influence of restricted alinement (sharp curves); and enforced changes in traffic conditions (at intersections on expressways). Many of these are investigated here for their effect. The list is not complete; any special geometric feature, traffic control device, or other element along the roadway which changes or influences the traffic pattern in any way should be considered. For instance, long underpasses on freeways, which give drivers a "tunnel effect," have been known to have an adverse effect on freeway traffic flow.

The main problem, after identifying a "below standard" location (i.e., one that is below the adopted level of service), is to adjust the design or conditions so that the desired level of service will be provided. In such adjustment, the first consideration must be that demand volumes never exceed service volumes for the adopted level of service, if that level is to be obtained throughout. Occasionally it may appear unfeasible to provide the basically-adopted level through some specific restriction within the section. Although traffic will continue to move without a backup as long as demand does not exceed capacity at any point, it will unavoidably move at a poorer level of service than that originally specified, at least for a limited distance. In the ultimate, however, when the traffic volume input exceeds the capacity of a roadway element, the roadway upstream from such a bottleneck becomes a storage area and calculated service volumes on that roadway section, within the area of influence of the bottleneck, have no meaning. The level of service and the highest attainable service volumes in this upstream zone

are independent of the geometric conditions at this location because they are bound to be governed by the capacity and operating restrictions at the bottleneck.

Intervehicular conflicts should not be such as to cause a sudden change in operating characteristics of the traffic stream. The necessity for lowered speeds or stopping during high volume conditions has a cumulative adverse effect on traffic flow. When traffic backs up from a bottleneck and is required to stop, the act of stopping creates a moving bottleneck in the form of minimum headways between vehicles departing from a stop condition. Therefore, it is important to recognize locations potentially critical in this respect during the design period and make design adjustments that will minimize the effect.

At-grade intersections, found occasionally on expressways though never on freeways, exemplify fixed traffic interruptions created of necessity. The maximum volume that can be carried through the uninterrupted portion of an expressway section between at-grade intersections can never exceed the capacity of the intersection approach at its downstream end, assuming that no other exits exist along the section. Neither can this portion carry more traffic than can be supplied by the next upstream intersection, provided there are no intermediate entrances. (It is, of course, possible for more to enter at the upstream end than can leave at the downstream end, thus producing a back-up in the section).

Lane Width and Lateral Clearance

Restrictive lane width and lateral clearances, discussed in Chapter Five, are not a consideration on most modern high-type freeways and expressways, because their design standards exceed those required for maximum capacity. Nevertheless, restrictions do exist on certain older freeways in such forms as 10- or 11-ft lanes, and abutments or other obstructions close to the traveled way. Table 9.2 presents adjustment factors which should be applied as multipliers to correct for any such limitations.

These adjustment factors, considered alone, indicate slightly greater adverse influence of clearance restrictions on 6- and 8-lane as compared to 4-lane freeways (due to the greater potential for restricted clearances between vehicles in adjacent lanes). However, because 6- and 8-lane freeways show fundamentally greater average capacities per lane under ideal capacities than do 4-lane, this slightly greater restrictive effect usually will not be apparent in practice, in final problem results.

Caution should be exercised in applying these adjustments to the situation sometimes found, particularly on older freeways, where the lateral clearance to a continuous median barrier in a narrow median, or sometimes to a right-side guardrail, is less than the 6 ft normally required for maximum capacity. As mentioned in Chapter Five, it appears that drivers regularly using a freeway rapidly adjust to the presence of such a continuous lateral obstruction, so that the adjustments shown in Table 9.2 become excessive. That is, although use of the full adjustment appears appropriate for "surprise"-type or dangerous elements, a lesser adjustment based on judgment may be suitable for continuous elements specifically designed and installed for traffic safety.

Trucks, Buses, and Grades on Freeways

Trucks and buses, being larger than passenger cars, take up more space, even in level terrain; hence, their influence on freeway service volumes and capacity must always be considered. Although their influence on level highways is relatively small, on grades it becomes significant.

As discussed in Chapter Five, the principal criterion for evaluating gradients on freeways and expressways, from a service volume and capacity standpoint, is their effect on the operating characteristics of trucks and buses. The overall effect of trucks and buses over an extended freeway section differs from that on any specific grade within that section.

Table 9.3a presents average generalized passenger car equivalents of trucks over extended lengths of freeways and expressways for various terrain conditions. These apply alike to all levels of service except level A, for which no overall equivalents are feasible. Normally, bus volumes are too small to warrant their separate consideration in these

overall reviews. However, separate approximate equivalents for buses also are given, for use where volumes are significant.

Table 9.3b provides general overall adjustment factors for conversion of mixed demand volumes of trucks and passenger cars over extended lengths of freeway into equivalent passenger vehicles per hour, based on these overall passenger car equivalents. These factors can be used in overall analyses of the capabilities of substantial lengths of freeway, which include downgrades and level portions as well as upgrades, but they should not be used for detailed analyses of specific individual grades. As just mentioned, in these overall computations separate consideration normally need not be given to buses. Where separate consideration appears necessary, however, Table 9.3b

is not appropriate. Rather, the equivalents for buses given in Table 9.3a should be used in conjunction with Table 9.6 to obtain separate adjustment factors.

Adjustment of service volumes and capacity to reflect the influence of trucks and buses on specific sustained upgrades is more selective. There is always a certain amount of platooning, or grouping of vehicles, even on level roads at relatively low volume levels. Often, a truck heads the platoon. When an upgrade is introduced under such conditions speeds are reduced and these platoons become more serious influences on service volumes and capacity. Their effect becomes more pronounced as volume increases. The frequency of platoons, and the speed at which they move, and hence the service volumes and capacity of the roadway, are

TABLE 9.2—COMBINED EFFECT OF LANE WIDTH AND RESTRICTED LATERAL CLEARANCE ON CAPACITY AND SERVICE VOLUMES OF DIVIDED FREEWAYS AND EXPRESSWAYS WITH UNINTERRUPTED FLOW

DISTANCE FROM TRAFFIC LANE EDGE TO OBSTRUCTION (FT)	ADJUSTMENT FACTOR,[a] W, FOR LANE WIDTH AND LATERAL CLEARANCE							
	OBSTRUCTION ON ONE SIDE OF ONE-DIRECTION ROADWAY				OBSTRUCTIONS ON BOTH SIDES OF ONE-DIRECTION ROADWAY			
	12-FT LANES	11-FT LANES	10-FT LANES	9-FT LANES	12-FT LANES	11-FT LANES	10-FT LANES	9-FT LANES
(a) 4-LANE DIVIDED FREEWAY, ONE DIRECTION OF TRAVEL								
6	1.00	0.97	0.91	0.81	1.00	0.97	0.91	0.81
4	0.99	0.96	0.90	0.80	0.98	0.95	0.89	0.79
2	0.97	0.94	0.88	0.79	0.94	0.91	0.86	0.76
0	0.90	0.87	0.82	0.73	0.81	0.79	0.74	0.66
(b) 6- AND 8-LANE DIVIDED FREEWAY, ONE DIRECTION OF TRAVEL								
6	1.00	0.96	0.89	0.78	1.00	0.96	0.89	0.78
4	0.99	0.95	0.88	0.77	0.98	0.94	0.87	0.77
2	0.97	0.93	0.87	0.76	0.96	0.92	0.85	0.75
0	0.94	0.91	0.85	0.74	0.91	0.87	0.81	0.70

[a] Same adjustments for capacity and all levels of service.

TABLE 9.3a—AVERAGE GENERALIZED PASSENGER CAR EQUIVALENTS OF TRUCKS AND BUSES ON FREEWAYS AND EXPRESSWAYS, OVER EXTENDED SECTION LENGTHS (INCLUDING UPGRADES, DOWNGRADES, AND LEVEL SUBSECTIONS)

LEVEL OF SERVICE		EQUIVALENT, E, FOR:		
		LEVEL TERRAIN	ROLLING TERRAIN	MOUNTAINOUS TERRAIN
A		Widely variable; one or more trucks have same total effect, causing other traffic to shift to other lanes. Use equivalent for remaining levels in problems.		
B through E	E_T, for trucks	2	4	8
	E_B, for buses[a]	1.6	3	5

[a] Separate consideration not warranted in most problems; use only where bus volumes are significant.

TABLE 9.3b—AVERAGE GENERALIZED ADJUSTMENT FACTORS FOR TRUCKS[b] ON FREEWAYS AND EXPRESSWAYS, OVER EXTENDED SECTION LENGTHS

PERCENTAGE OF TRUCKS, P_T	FACTOR, T, FOR ALL LEVELS OF SERVICE		
	LEVEL TERRAIN	ROLLING TERRAIN	MOUNTAINOUS TERRAIN
1	0.99	0.97	0.93
2	0.98	0.94	0.88
3	0.97	0.92	0.83
4	0.96	0.89	0.78
5	0.95	0.87	0.74
6	0.94	0.85	0.70
7	0.93	0.83	0.67
8	0.93	0.81	0.64
9	0.92	0.79	0.61
10	0.91	0.77	0.59
12	0.89	0.74	0.54
14	0.88	0.70	0.51
16	0.86	0.68	0.47
18	0.85	0.65	0.44
20	0.83	0.63	0.42

[b] Not applicable to buses where they are given separate specific consideration; use instead Table 9.3a in conjunction with Table 9.6.

TABLE 9.4—PASSENGER CAR EQUIVALENTS OF TRUCKS ON FREEWAYS AND EXPRESSWAYS, ON SPECIFIC INDIVIDUAL SUBSECTIONS OR GRADES

GRADE (%)	LENGTH OF GRADE (MI)	PASSENGER CAR EQUIVALENT, E_T									
		LEVELS OF SERVICE A THROUGH C FOR:					LEVELS OF SERVICE D AND E (CAPACITY) FOR:				
		3% TRUCKS	5% TRUCKS	10% TRUCKS	15% TRUCKS	20% TRUCKS	3% TRUCKS	5% TRUCKS	10% TRUCKS	15% TRUCKS	20% TRUCKS
0–1	All	2	2	2	2	2	2	2	2	2	2
2	¼–½	5	4	4	3	3	5	4	4	3	3
	¾–1	7	5	5	4	4	7	5	5	4	4
	1½–2	7	6	6	6	6	7	6	6	6	6
	3–4	7	7	8	8	8	7	7	8	8	8
3	¼	10	8	5	4	3	10	8	5	4	3
	½	10	8	5	4	4	10	8	5	4	4
	¾	10	8	6	5	5	10	8	5	4	5
	1	10	8	6	5	6	10	8	6	5	6
	1½	10	9	7	7	7	10	9	7	7	7
	2	10	9	8	8	8	10	9	8	8	8
	3	10	10	10	10	10	10	10	10	10	10
	4	10	10	11	11	11	10	10	11	11	11
4	¼	12	9	5	4	3	13	9	5	4	3
	½	12	9	5	5	5	13	9	5	5	5
	¾	12	9	7	7	7	13	9	7	7	7
	1	12	10	8	8	8	13	10	8	8	8
	1½	12	11	10	10	10	13	11	10	10	10
	2	12	11	11	11	11	13	12	11	11	11
	3	12	12	13	13	13	13	13	14	14	14
	4	12	13	15	15	14	13	14	16	16	15
5	¼	13	10	6	4	3	14	10	6	4	3
	½	13	11	7	7	7	14	11	7	7	7
	¾	13	11	9	8	8	14	11	9	8	8
	1	13	12	10	10	10	14	13	10	10	10
	1½	13	13	12	12	12	14	14	13	13	13
	2	13	14	14	14	14	14	15	15	15	15
	3	13	15	16	16	15	14	17	17	17	17
	4	15	17	19	19	17	16	19	22	21	19
6	¼	14	10	6	4	3	15	10	6	4	3
	½	14	11	8	8	8	15	11	8	8	8
	¾	14	12	10	10	10	15	12	10	10	10
	1	14	13	12	12	11	15	14	13	13	11
	1½	14	14	14	14	13	15	16	15	15	14
	2	14	15	16	16	15	15	18	18	18	16
	3	14	16	18	18	17	15	20	20	20	19
	4	19	19	20	20	20	20	23	23	23	23

functions of (a) the number of slow vehicles, (b) the rate of grade, and (c) the length of grade. If volumes are low, the grade is short, and there are few trucks, there is relatively small probability that any given vehicle will encounter trucks on the grade. If the grade is longer, there is a greater probability that trucks will be encountered on the grade. Also, if the grade is steeper (and thus trucks slower), trucks will be on the grade a greater portion of the time. As indicated earlier in Chapter Five, only limited research linking these variables has been reported; much remains to be learned.

It is assumed that freeway grades of less than 2 percent which are less than one-half mile long will have little effect on operations. Grades in the neighborhood of 2 percent may produce queues, but the queues will move fast enough so that high rates of flow can be maintained; significant accumulations of vehicles are not likely to develop if the grade is less than one-half mile long. Nevertheless, the speed of trucks will be substantially reduced and the exposure to rear-end collisions increased.

On sustained grades, normally the right-hand lane will be pre-empted by trucks, with operating speeds in this lane controlled by the climbing ability of the trucks; passenger cars will avoid this lane if conditions are better in the remaining lanes. If all trucks are traveling in the right-hand lane, then all passenger cars (or all vehicles that can maintain the adopted operating levels achieved on level terrain) must be in the remaining lanes if no passenger car is to be influenced adversely by trucks. It follows that, to maintain a level of service to passenger cars on the grade equal to that on level terrain, it will be necessary to add a climbing lane whenever volumes in the remaining lanes increase to the point where passenger car speeds would otherwise fall below that for the adopted level of service. If it is not possible to keep all passenger cars out of lane 1, or if trucks travel in other than lane 1, it may be necessary to add additional climbing lanes to maintain the desired level of service.

In practice, because of economic factors it may not always be feasible to provide the desired level of service at every point. If, in any case, for a selected level of service, the demand volume anticipated for the design hour exceeds the total service volume, the decision must be made whether to provide an additional lane on the upgrade or to accept a lower level of service through this critical section. This decision must be primarily an economic one, usually between level B and level C on rural freeways, and between C and D on urban freeways. However, in no case can the demand volume exceed the maximum service volume for level E (capacity) on the grade, if back-ups are to be avoided. Where such a situation would otherwise exist, it is essential that an additional upgrade lane be provided to prevent breakdown into level F and storage of flow on the approach to the grade.

Table 9.4 presents the detailed passenger car equivalency factors which represent the extent to which capacity and service volumes will be adversely affected, on the average across all lanes, on individual sustained freeway upgrades where an additional truck climbing lane is not introduced.

As was mentioned in Chapter Five, intercity bus volumes usually are quite low in freeway traffic flows. This fact, coupled with their relatively good performance on most typical grades, makes varying adjustment for buses unnecessary in most cases; the general equivalent of 1.6, mentioned in Chapter Five, can be used for most purposes. However, where the grade involved is long and steep, and/or volumes of buses are heavy, special consideration may be desirable. Table 9.5 presents passenger car equivalents for buses under such conditions.

In most practical applications, as described in the later procedural section of this chapter, the equivalency factors for trucks and buses presented in Tables 9.4 and 9.5 are not used directly. Rather, they are used to select appropriate truck adjustment factors from Table 9.6, which considers both passenger car equivalency and percentage of trucks or buses in the traffic stream, as related to grade characteristics; these are applied as multipliers.

On most flat to intermediate downgrades passenger car equivalents and truck factors can be considered the same as those on level ground without appreciable error. On heavy downgrades, however, where trucks descend

in a low gear for safety, special consideration may need to be given. If an average speed can be determined for downgrade trucks, then by reference to the truck performance curves in Chapter Five the downgrade performance can be approximately related to equivalent upgrade performance for which adjustments are available.

Weaving Areas

Operations at weaving areas have been analyzed in Chapter Seven for the fundamental case where two or more important through roadways join for a certain distance then diverge again into separate roadways, as well as in Chapter Eight for those situations where the weaving between an on-ramp junction and a successive off-ramp junction is being analyzed. Usually, on a freeway, any weaving sections involved are relatively critical locations which must be analyzed for their overall effect on the roadway section. Although traffic flow may be maintained, unsatisfactory operating conditions at weaving areas will greatly affect the operating characteristics of the main roadways upstream and downstream from the weaving area for substantial distances. It is essential, therefore, that the weaving section design be de-veloped so that the adopted level of service can be attained at all points. As mentioned earlier, this does not necessarily mean the same speed throughout; slightly slower speeds are acceptable through critical locations, including weaving sections.

Direct reference should be made to Chapters Seven and Eight for the procedural steps to be followed in computing service volumes and capacities of weaving sections found along any freeway section under consideration.

Ramp Terminals

The operating characteristics of on- and off-ramp junctions have been analyzed in detail in Chapter Eight. On freeways and most other expressways the location of these ramps and the traffic demand on them are the chief determinants of the varying demand volumes along the through roadway section.

The problem at on-ramps is primarily one of blending into one flow traffic introduced from two sources. Intervehicular conflicts are numerous, and the geometric layout at the junction, as well as that junction's relation to other nearby junctions, is extremely important. Aside from the effects of any

TABLE 9.5—PASSENGER CAR EQUIVALENTS OF INTERCITY BUSES ON FREEWAYS AND EXPRESSWAYS, ON SPECIFIC INDIVIDUAL SUBSECTIONS OR GRADES

| GRADE[a] (%) | PASSENGER CAR EQUIVALENT,[b] E_B | |
	LEVELS OF SERVICE A THROUGH C	LEVELS OF SERVICE D AND E (CAPACITY)
0–4	1.6	1.6
5[c]	4	2
6[c]	7	4
7[c]	12	10

[a] All lengths.
[b] For all percentages of buses.
[c] Use generally restricted to grades over 1/2 mile long.

adverse geometrics, on-ramps may create two conflicts with the maintenance of the adopted level of service of a roadway section. First, the additional ramp traffic may cause operational changes in and/or temporary overloading of the right-hand lane at the merge. Second, the additional ramp volume may change the operating conditions across the entire roadway downstream from the on-ramp.

The problem at off-ramps is primarily one of dividing a single flow of traffic into two paths, one continuing through and the other exiting. A conflict area may be created due to (a) high demand for use of the right-hand lane, and consequent speed reduction, caused by exiting vehicles superimposed on the through flow in lane 1, and (b) backup from the off-ramp onto the main roadway proper. Most of these exit ramp problems

TABLE 9.6—ADJUSTMENT FACTORS[a] FOR TRUCKS AND BUSES ON INDIVIDUAL ROADWAY SUBSECTIONS OR GRADES ON FREEWAYS AND EXPRESSWAYS (INCORPORATING PASSENGER CAR EQUIVALENT AND PERCENTAGE OF TRUCKS OR BUSES)[b]

PASSENGER CAR EQUIVALENT, E_T OR E_B[c]	TRUCK ADJUSTMENT FACTOR T_c OR T_L (B_c OR B_L FOR BUSES)[d]														
	PERCENTAGE OF TRUCKS, P_T (OR OF BUSES, P_B) OF:														
	1	2	3	4	5	6	7	8	9	10	12	14	16	18	20
2	0.99	0.98	0.97	0.96	0.95	0.94	0.93	0.93	0.92	0.91	0.89	0.88	0.86	0.85	0.83
3	0.98	0.96	0.94	0.93	0.91	0.89	0.88	0.86	0.85	0.83	0.81	0.78	0.76	0.74	0.71
4	0.97	0.94	0.92	0.89	0.87	0.85	0.83	0.81	0.79	0.77	0.74	0.70	0.68	0.65	0.63
5	0.96	0.93	0.89	0.86	0.83	0.81	0.78	0.76	0.74	0.71	0.68	0.64	0.61	0.58	0.56
6	0.95	0.91	0.87	0.83	0.80	0.77	0.74	0.71	0.69	0.67	0.63	0.59	0.56	0.53	0.50
7	0.94	0.89	0.85	0.81	0.77	0.74	0.70	0.68	0.65	0.63	0.58	0.54	0.51	0.48	0.45
8	0.93	0.88	0.83	0.78	0.74	0.70	0.67	0.64	0.61	0.59	0.54	0.51	0.47	0.44	0.42
9	0.93	0.86	0.81	0.76	0.71	0.68	0.64	0.61	0.58	0.56	0.51	0.47	0.44	0.41	0.38
10	0.92	0.85	0.79	0.74	0.69	0.65	0.61	0.58	0.55	0.53	0.48	0.44	0.41	0.38	0.36
11	0.91	0.83	0.77	0.71	0.67	0.63	0.59	0.56	0.53	0.50	0.45	0.42	0.38	0.36	0.33
12	0.90	0.82	0.75	0.69	0.65	0.60	0.57	0.53	0.50	0.48	0.43	0.39	0.36	0.34	0.31
13	0.89	0.81	0.74	0.68	0.63	0.58	0.54	0.51	0.48	0.45	0.41	0.37	0.34	0.32	0.29
14	0.88	0.79	0.72	0.66	0.61	0.56	0.52	0.49	0.46	0.43	0.39	0.35	0.32	0.30	0.28
15	0.88	0.78	0.70	0.64	0.59	0.54	0.51	0.47	0.44	0.42	0.37	0.34	0.31	0.28	0.26
16	0.87	0.77	0.69	0.63	0.57	0.53	0.49	0.45	0.43	0.40	0.36	0.32	0.29	0.27	0.25
17	0.86	0.76	0.68	0.61	0.56	0.51	0.47	0.44	0.41	0.38	0.34	0.31	0.28	0.26	0.24
18	0.85	0.75	0.66	0.60	0.54	0.49	0.46	0.42	0.40	0.37	0.33	0.30	0.27	0.25	0.23
19	0.85	0.74	0.65	0.58	0.53	0.48	0.44	0.41	0.38	0.36	0.32	0.28	0.26	0.24	0.22
20	0.84	0.72	0.64	0.57	0.51	0.47	0.42	0.40	0.37	0.34	0.30	0.27	0.25	0.23	0.21
21	0.83	0.71	0.63	0.56	0.50	0.45	0.41	0.38	0.36	0.33	0.29	0.26	0.24	0.22	0.20
22	0.83	0.70	0.61	0.54	0.49	0.44	0.40	0.37	0.35	0.32	0.28	0.25	0.23	0.21	0.19
23	0.82	0.69	0.60	0.53	0.48	0.43	0.39	0.36	0.34	0.31	0.27	0.25	0.22	0.20	0.19
24	0.81	0.68	0.59	0.52	0.47	0.42	0.38	0.35	0.33	0.30	0.27	0.24	0.21	0.19	0.18
25	0.80	0.67	0.58	0.51	0.46	0.41	0.37	0.34	0.32	0.29	0.26	0.23	0.20	0.18	0.17

[a] Computed by 100/(100 − P_T + $E_T P_T$), or 100/(100 − P_B + $E_B P_B$), as presented in Chapter Five. Use this formula for larger percentages.
[b] Used to convert equivalent passenger car volumes to actual mixed traffic; use reciprocal of these values to convert mixed traffic to equivalent passenger cars.
[c] From Table 9.4 or Table 9.5.
[d] Trucks and buses should not be combined in entering this table where separate consideration of buses has been established as required, because passenger car equivalents differ.

can be solved by using adequate geometric design features—signing, transition areas, additional lanes or ramps, and vehicular storage—provided ramps are not too closely spaced and the surrounding highway system is basically adequate to absorb the load.

Thus, the traffic volume on a freeway or other expressway changes at every entrance and exit ramp, with corresponding variations in roadway operating conditions. Because it is impossible to design a highway so that demand volumes remain constant, the most critical point of analysis will be where volumes are a maximum, including the merge point just downstream from a ramp entrance and the diverge point just upstream from a ramp exit.

Direct reference should be made to Chapter Eight for procedures to be followed in determining service volumes and capacities of ramp terminals. It includes recommended maximum service volumes allowable at these previously mentioned critical points if a desired level of service on the through lanes is to be maintained.

Alinement

Adverse alinement is relatively unusual on freeways and expressways. Where it exists, however, its effect is reflected in the lowered average highway speeds resulting. As previously mentioned, only approximate data are available regarding the influence of these lowered average highway speeds on freeway operating speeds and volumes carried. These are incorporated directly into the computational criteria that follow.

Traffic Interruptions (Intersections at Grade)

Intersections at grade, absent by design on full freeways, are permissible on expressways under certain conditions. They are the key identifying features distinguishing expressways from full freeways, although in addition some expressways have only partial control of access between intersections. The analysis of signalized intersection approach capacities and service volumes has been discussed in Chapter Six; the procedures there described are generally applicable to at-grade intersections on expressways. Basically, the capacity of an intersection approach on

an expressway sets the maximum attainable service volume upstream from the intersection, at least to the next intersection, but there are occasional exceptions on lower-class expressways where access is only partially controlled and other access points exist. Downstream from the intersection, the maximum traffic demand will be limited to the capacity of the through lanes of the intersection approach plus additional traffic entering by turns from the cross road or other access points.

Rural expressways generally have relatively few such intersections, and those which are present usually serve very light traffic volumes, under stop sign control. Grade separations rather than signals are typically provided at the more heavily traveled crossings. On such highways operating conditions may approach those for full freeways for capacity and service volume determination purposes, provided that a reasonable degree of access control exists between intersections. Actually, at typical levels of service on such a rural expressway the only major difference is the potential accident hazard.

Where a significant degree of control of access has not been maintained and "ribbon" or "strip" business development has occurred along the roadside, expressway criteria should no longer be applied; the highway should be analyzed by the methods given in Chapter Ten for ordinary highways.

Suburban and urban expressways, on the other hand, have a somewhat different connotation. Typically they are very high-type arterials with all or nearly all midblock access points (such as from "ribbon" development) eliminated, but with relatively frequent signalized intersections, usually interconnected for progressive operation. This signalization may well be the only interruption-producing feature on the expressway.

Signalization will obviously produce a capacity loss, as compared to uninterrupted flow, on a per-hour basis. The influence of signalization on service volumes, however, depends on the type of operation desired. If occasional stops at signals can be tolerated and the percentage of red time is relatively small, the approaches and exits often can be

widened sufficiently to accommodate, during the green time, as much traffic as the roadway ahead can absorb on a continuous basis at a reasonable operating level. Overall level of service will be reduced somewhat due to the occasional stops, but more effective use of the midblock sections results. Such widening is sometimes found on urban expressways.

Theoretically, where near-perfect progression is attained on expressways, then, on a per-hour-of-green basis, it is possible for full freeway capacity and service volume determination criteria to apply directly at all levels. That is, the vehicles in the progressive platoons move just as they would on freeways; the only difference is that enforced gaps exist between platoons. Obviously, the actual per-hour capacity and service volumes of the system, obtained by applying the G/C ratio to the per-hour-of-green values, will be less than the equivalent freeway values, the reduction being proportional to the percentage of red time. Here, where all cars are kept moving, level of service will be uniform throughout, but the time between moving platoons (approximately equal to the sum of the signal red and yellow times) is entirely lost to through traffic at every level, along the midblock sections just as much as at the intersection. Widening at intersections will serve primarily as a "safety factor." The special characteristics of near-perfect progressions are discussed in more detail in Chapter Ten.

In general, if an expressway has signals spaced more than 1 mile apart, an attainable speed limit of at least 45 mph between signals at low volumes, and reasonable control of access between signals, it is considered acceptable to base determination of its capabilities on full freeway criteria for uninterrupted flow. This, in effect, assumes that the signals will stop relatively few vehicles. At poorer levels of service, this assumption will become relatively invalid and the capacity of controlling signalized intersections, as related to the lowered operating speeds resulting from stops, may have to be taken into account in establishing capabilities.

Where signalized intersections are closer than 1 mile apart and good progression does not exist, where speed limits are 40 mph or below, or where higher speed limits seldom can be attained because of friction due to ribbon development, the expressway should be analyzed as an urban arterial by the methods in Chapter Ten.

COMPUTATION PROCEDURES FOR FREEWAYS AND EXPRESSWAYS

In Chapter Four the generalized procedure for determination of level of service for uninterrupted flow conditions was described. In this section, capacity and service volume determination procedures, and the level of service procedure previously described, are applied first to basic freeway and expressway sections, then to combined sections composed of several different elements.

The first step shown in the general procedures for all highway types involves the subdivision of the roadway under consideration into subsections having reasonably uniform conditions from the standpoint of capacity. In the case of modern freeways, which are designed to high uniform standards, there are many situations, particularly in rural areas, where such subdivision is not necessary to determine the capacity, service volumes, or level of service of even relatively long sections. Only where a ramp junction, weaving section, significant grade, or other special design feature is present will subsection analyses be necessary. On older freeways there may be other restricting elements, such as substandard alinement with sharp curvature, which require separate subsection analysis.

Basic Uniform Freeway and Expressway Sections

The initial procedures described here apply to a simple basic uniform freeway section without entrance or exit points; it may range from a few hundred feet to many miles long. As used here, "freeway capacity" and "freeway service volumes" will refer to the total volumes in one direction. Average "by lane" capacities and service volumes can be obtained by dividing "total in one direction" values by the number of lanes. However, such average values should be used with caution, because they do not reflect the

actual distribution of traffic by lanes; their use can cause undesirable misinterpretations.

As discussed in Chapter Four, operating speed and service or demand volume/capacity ratio (v/c ratio) are the basic measures used in making level of service determinations on freeways and expressways. The limitations defining the several levels of service have been described at the start of this chapter and summarized in Table 9.1, which serves as the base for most computations.

Figure 9.1 presents these basic relationships graphically. Although similar in appearance to the typical operating speed-volume chart presented earlier in Figure 3.35, the service or demand volume/capacity ratio is substituted for the absolute volume along the abscissa. Hence, it can be applied to any highway, of any number of lanes, for which the capacity can be determined, regardless of whether or not the associated conditions are ideal. Occasionally, problems involving interpolation can be handled more conveniently through use of the figure than by use of the basic Table 9.1. The figure is also convenient for quick visual analyses, or checks of results. The basic limiting values of operating speed and v/c ratio which identify the several levels of service are shown on the chart.

Direct determination of capacities of actual freeways having less-than-ideal conditions involves simply application of one or more adjustment factors to the basic value under ideal conditions of 2,000 passenger cars per lane per hour times the number of
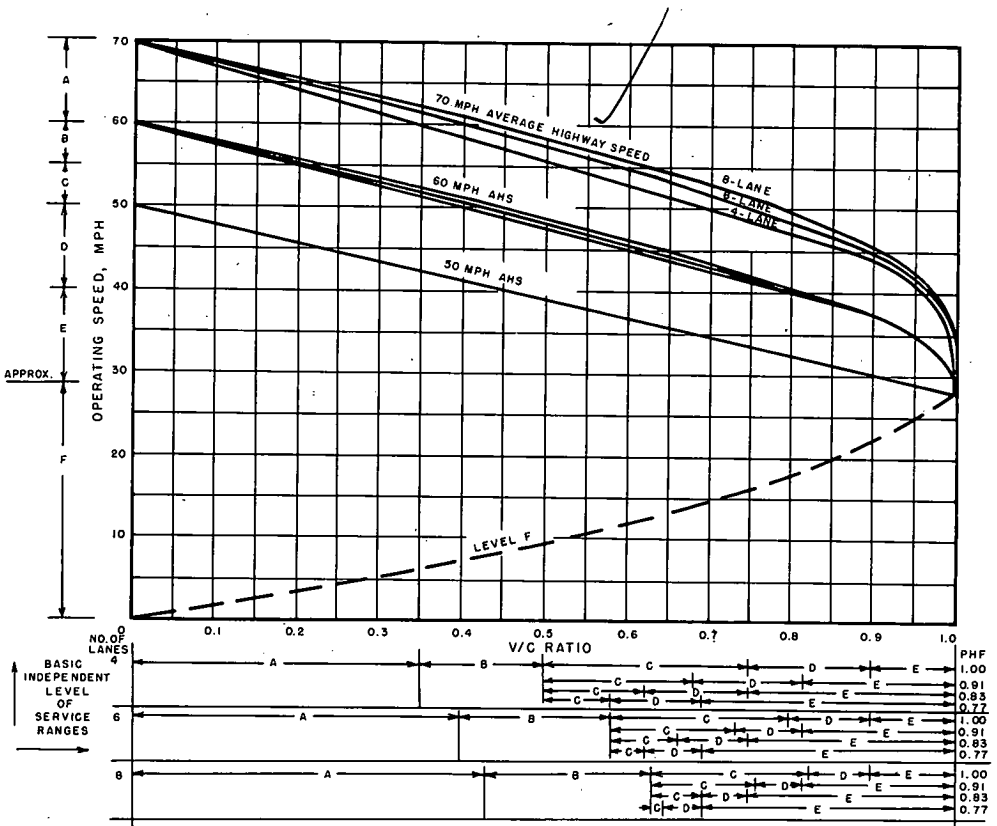
Figure 9.1. Relationships between v/c ratio and operating speed, in one direction of travel, on freeways and expressways, under uninterrupted flow conditions.

lanes, as tabulated for level E in Table 9.1. Determination of service volumes and of levels of service is somewhat more involved, making use of the operating speed-v/c ratio relationships in Table 9.1 (or Fig. 9.1). The procedures follow.

CAPACITY (TOTAL FOR ONE DIRECTION) UNDER PREVAILING CONDITIONS

Determine directly, by the standard method for uninterrupted flow. This involves multiplying the appropriate level E volume value in Table 9.1 (that is, the appropriate multiple of 2,000 passenger cars per lane per hour) by the applicable adjustment factors among those included in Tables 9.2 and 9.6. Often, only the adjustment for trucks will be required, because modern freeway geometrics are not often substandard from a capacity standpoint. The truck adjustment used must be that for capacity, not for a level of service.

$$c = 2,000 \; N \; W \cdot T_c$$

in which

$c =$ capacity (mixed vehicles per hour; total for one direction);

$N =$ number of lanes (in one direction);

$W =$ adjustment for lane width and lateral clearance from Table 9.2. (This adjustment must be used with discretion in freeway capacity computations. See also possible shoulder adjustment in Chapter Five); and

$T_c =$ truck factor *at capacity,* from Table 9.3b for overall highway sections or Table 9.6 for specific individual grades. (Intercity bus factor, B_c, may be applied separately; see text).

SERVICE VOLUMES (TOTAL FOR ONE DIRECTION)

Several different procedures are available for use in determining the service volume for a given level of service on actual highways having less-than-ideal conditions. Selection of the appropriate method depends on the data already at hand in any particular case. Regardless of which method is employed, it is important to check the result by means of Table 9.1 (or Fig. 9.1) to confirm that *both* the operating speed and the volume criteria for the desired level of service are met, with due consideration for the prevailing average highway speed.

Computed Directly from Capacity under Ideal Conditions.—The procedure is a modification of that described previously for capacity. Again 2,000 passenger cars per lane per hour is multiplied by the number of lanes and appropriate adjustments. However, the adjustment for trucks must be that for the appropriate level of service, rather than that for capacity. The appropriate v/c ratio must be applied for the level of service desired and the number of lanes provided in one direction. Where ideal alinement is not present, meaning that the average highway speed is below 70 mph, use of the applicable v/c ratio as given in Table 9.1, rather than the basic limiting ratio, will help to assure a result in balance with the operating speed limitations. Or, the appropriate average highway speed curve in Figure 9.1 can be referred to, to achieve a balance. In levels C and D, selection of the v/c ratio also involves consideration of the peak-hour factor as a multiplier.

$$SV = 2,000 \; N \frac{v}{c} \; W \cdot T_L$$

in which

$SV =$ service volume (total for one direction);

$N =$ number of lanes (mixed vehicles per hour, in one direction);

$v/c =$ volume to capacity ratio, obtained from Table 9.1 (or Fig. 9.1);

$W =$ adjustment for lane width and lateral clearance, from Table 9.2 (shoulder adjustment may be necessary, see Chapter Five); and

$T_L =$ Truck factor at given level of service, from Table 9.3b for overall highway sections, or Table 9.6 for specific individual grades (Intercity bus factor, B_L, may be applied separately; see text).

Confirm attainment of desired level of service by using Figure 9.1 to check the result-

ing operating speed, for the given average highway speed, to make sure that it meets requirements for that level.

Computed from Maximum Service Volume for Ideal Conditions.—This procedure, which is suitable only where the alinement is ideal (that is, average highway speed is 70 mph), is identical to the foregoing procedure except that the maximum service volume for the level of service (and PHF, for levels C and D) desired, obtained from Table 9.1, is used in place of the basic value adjusted by a *v/c* ratio.

$$SV = MSV \ W \ T_L$$

in which *MSV* is the maximum service volume, in passenger cars per hour, for the appropriate number of lanes (and PHF, if appropriate), from Table 9.1; and *SV*, *W*, and T_L are defined as before.

Confirm attainment of desired level of service by using Figure 9.1 to check the resulting operating speed. *Caution: Use of this method is not appropriate where restricted average highway speeds exist, because it does not make use of the v/c ratio, in which the influence of average highway speed restrictions is incorporated.*

Computed from Capacity under Prevailing Conditions.—Multiply the capacity obtained under prevailing conditions by the *v/c* ratio obtained from Table 9.1 (or Fig. 9.1) for the appropriate number of lanes and level of service (and PHF, if level C and D) desired. As for ideal conditions, consider use of a working *v/c* ratio where average highway speed is restricted. Also, convert the truck adjustment, if used, to that for the level of service involved, rather than capacity.

$$SV = c\frac{v}{c} \ \frac{T_L}{T_c}$$

in which *c* is the capacity (mixed vehicles per hour, total for one direction) as computed under prevailing conditions, and *v/c*, T_L, and T_c as defined as before.

Confirm attainment of desired level of service by checking the resulting operating speed, from Figure 9.1, for the given average highway speed.

Determined from Level of Service Limits. —In the design of a new freeway, where a specified level of service has been established in advance, service volumes in passenger cars per hour can be read directly from Table 9.1, provided conditions are largely ideal (as they might well be on a freeway carrying only a few trucks). Where alinement is not ideal (average highway speed is less than 70 mph) or conditions are otherwise less than ideal, Table 9.1 can be used to determine the limiting *v/c* ratio. From the controlling ratio, the service volume can be determined once capacity is computed. (Fig. 9.1 can also be used.)

Where a proposed design is already under consideration, the *v/c* ratio here obtained can be compared with that for the proposed design to determine its adequacy.

LEVEL OF SERVICE

Determination of the level of service provided by any freeway or expressway design, existing or proposed, under uninterrupted flow operation while accommodating a given demand volume, is often the problem at hand. This can be done approximately by inspection of Table 9.1, if operating speed, volume, peak-hour factor, and average highway speed are known, and the influence of trucks can be neglected. However, a refined computation considering trucks and peaking characteristics involves complications which make a partially "trial-and-error" solution unavoidable. Although knowledge of the level of service is needed in order to choose the truck factor and to establish whether or not consideration of the peak-hour factor is required, it is the unknown. Therefore, a level must be assumed in advance, usually by inspection of Table 9.1, and recomputations carried out if the results prove the assumption incorrect.

The steps are as follows:

(a) Establish a "base volume" for level of service determination through the same procedure as described in the preceding section on "Service Volumes; Computed Directly from Capacity under Ideal Conditions," except that no *v/c* ratio is applied. ("Base volume" for the prevailing conditions differs from capacity only in

that the truck factor is that for the assumed level of service, rather than for capacity.)

$$\text{Base volume} = 2,000 \ N \ W \ T_L$$

in which N and W are as before and T_L is the assumed truck factor.

(b) Divide the average demand volume by the "base volume" obtained in (a) to determine the approximate v/c ratio. (Conversion of demand volume to equivalent passenger cars is not necessary, inasmuch as use of the truck factor in Step (a) has converted the base to mixed traffic.)

(c) Reinspect Table 9.1 or Figure 9.1 if operating speed was known in advance, to establish level of service from controlling factor, operating speed or basic v/c ratio, with due consideration of the PHF applicable to the level assumed.

If operating speed was not known, enter Table 9.1 for the appropriate conditions and determine operating speed. Or, enter Figure 9.1 on the v/c ratio scale, select the appropriate curve for the number of lanes in one direction and the average highway speed under consideration, and read the operating speed. Establish level of service from controlling factor, operating speed or basic v/c ratio.

(d) Recompute, using revised choice of truck factor and PHF, based on different assumed level of service, if initial assumption proves incorrect.

Combined Analysis of Elements Composing Freeway and Expressway Sections

As previously mentioned, the procedures described so far, which apply to a single uniform roadway section, will suffice to establish the characteristics of a long section of freeway, provided the section is free of any restrictive elements. However, in most situations there will be a variety of elements, such as grades, ramp junctions, weaving sections, or sections with differing number of lanes, along any freeway segment of significant length, which produce nonuniform characteristics. Balanced operation of the complete freeway section demands the relating of the operation of each of these separate elements,

as previously determined, to the overall operation of the section.

First, the matter of units must be considered. Whereas the basic maximum freeway volume values for ideal conditions contained in this chapter are in terms of passenger cars per hour, it will be noted that the procedures in Chapters Seven and Eight for weaving section and ramp junction operation are basically in terms of average mixed traffic (although the actual weaving movement in Chapter Eight is converted to equivalent passenger cars). This seeming incongruity results from the differing bases on which the several research studies involved were developed.

Because it is relatively easy to convert the basic "through freeway" procedures to mixed traffic, as done in the procedures just presented, but rather unfeasible to convert the ramps procedures to equivalent passenger cars, it is recommended that actual problems involving a series of freeway elements be carried out in terms of mixed traffic. This procedure has the added advantage of discussing actually existing traffic volumes, rather than artificial equivalent volumes. In any case, caution should be exercised in all comparisons with other analysis results, to confirm consistency of units.

Typically, one of two problems exists. Either an existing freeway requires analysis to determine whether or not it has "weak links" which have lower traffic-carrying capabilities than the remainder, or a new freeway is being designed with the goal of fully balanced design. In the former case, geometrics and demand volumes will usually be known and levels of service are required. In the latter, demand volumes and level of service normally will be specified, and geometrics are required. In practice, however, trial designs will often be developed so that the actual computations will parallel those for existing highways.

Certainly, an all-important goal of any new design is to create a balanced level of service throughout. It is important to recognize, however, that where total balance is impossible, a freeway designed generally to provide a certain basic level of service but having one or two restricted subsections somewhat below that level, will nevertheless

provide far better service than a similar free-way with many such restricted subsections, as long as demand does not exceed capacity at any point. That is, the finding that attainment of the desired level appears unfeasible at one or two points should not be used as justification for overall lowering of standards.

Generally speaking, the type of problem likely to be encountered can best be demonstrated by means of actual examples rather than by discussion; this is done in the section on typical problem solutions, which follows. However, certain observations regarding the feasibility of overall or "weighted" values deserve mention here. As long as fully balanced design is attained, "weighting" is, of course, superfluous, inasmuch as the level is identical at all points. Nevertheless, the fact remains that many existing freeways are not of balanced design and cannot be balanced without major reconstruction. On such freeways, as well as on the previously mentioned new designs of less than perfect balance, therefore, for traffic operations, planning, and economic study purposes it is desirable to develop some general measure of average overall performance.

Any highway section, including a freeway, can have only one capacity between a particular point of entrance and the next exit; namely, the capacity of the most restrictive subsection within that section. Between any two terminal points A and B, with entrances and exits at intermediate points, the controlling capacity is less well defined. Here, there will still be a limiting capacity somewhere between A and B, but it may not affect all traffic at all points, depending on the pattern of intermediate entrance and exit demands. It can be seen, therefore, that choice of terminal points will have an important bearing on ability to identify a controlling capacity. Where, as in a suburb-to-downtown case, entering traffic predominates, such identification is easier than in the case where entrance and exit volumes fluctuate randomly with no such overall trend. In any case, a "weighted capacity" value would be largely meaningless, except for specialized interpretations over relatively consistent sections.

Similarly, a "weighted service volume" is useful only for generalizations regarding the relationship of demand to overall capabilities, because as long as capacity is not exceeded at any point along a section the assigned demand volume will be accommodated throughout, regardless of how this demand volume relates to specified service volume limits. Level of service may, of course, vary substantially.

On the other hand, a "weighted level of service," as described earlier in the general procedures presented in Chapter Four, often has considerable value as a measure of the overall performance of a freeway section of some length. This weighting of operating speeds and v/c ratios, the indicators of levels of service, is done in terms of the relative lengths of the sections involved, or their relative influence areas. Wherever precise weighting is involved, therefore, at least approximate length increments, operating speeds, and v/c ratios must be known.

In practice, the procedure is not always as simple as is demonstrated in the sample problem in Chapter Four. There, only easily identified basic roadway sections were involved; in such cases, operating speeds and v/c ratios are either at hand or can be determined without difficulty. Where, on the other hand, elements such as ramp junctions and weaving sections are included in the section under consideration, the problem is more difficult because only approximate operating speeds, which differ from through roadway speeds, are identified with several levels of service, and v/c ratios are not directly used. In the extreme, where at-grade intersections are involved, no speed measure exists at all. In such cases direct weighted averages would be incorrect, even if operating speeds and v/c ratios were known, because differing scales would be involved.

Similarly, for some freeway elements lengths of section are obvious. For others, however, the influence area for use in weighting is not readily apparent. In particular, ramp junctions do not have "lengths" as such. Interpretation of the material in Chapter Eight indicates that, as a rule of thumb, a ramp junction influence distance totaling 3,000 ft can be assumed. In the on-ramp case this is composed of about 500 ft upstream and 2,500 ft downstream; in the off-

A rural freeway intersecting a local highway.

ramp case it consists of 2,500 ft upstream and 500 ft downstream. (Where an overlap occurs, the poorer of the two sets of operating conditions should be considered as applicable to the overlap area.) For weaving sections, an influence distance of about 1,000 ft in addition to the section length (500 ft upstream and 500 ft downstream) may be assumed. Seldom, on freeways, are significant lateral clearance restrictions encountered. Where they are, their influence distances should be determined as described in Chapter Ten for ordinary multilane highways.

In practice, precise determination of the average overall level of service over a long section of roadway is seldom required.

Rather, a general idea of the level is needed. Therefore, determination of a weighted level by precise methods is usually limited to cases where operating speeds, v/c ratios, and lengths are readily available, and the level of service scales are uniform throughout. The procedure follows.

First, compute the weighted average of the operating speeds obtained for each subsection, by multiplying the length of each section by its operating speed, summing the several results, and dividing by the overall length. Similarly, compute the weighted average of the v/c ratios, in the same manner if the number of lanes is the same throughout. However, where the number of

lanes varies, the limiting v/c ratios will vary. Hence, the weighted average for each width category should be developed separately. Finally, using these average operating speed and v/c ratio values, determine the resulting overall average level of service for the section, using Table 9.1 (or Fig. 9.1).

As a check of the feasibility of the average operation thus developed, it is necessary to relate the weighted average operating speed to the most critical v/c ratio to make sure that capacity is not exceeded at any point. Or, if a predetermined controlling level of service has been established, use this check to assure that this limit is not exceeded.

In other cases, where ramp junctions, weaving sections, and at-grade intersections are involved, an approximate weighting by inspection of the letter designations is adequate for most purposes, and more feasible than attempting to develop and combine values based on differing criteria. A graphical plot, as shown in Example 9.8b, is a convenient aid. In effect, this is a substitute for the impossible task of weighting measures taken from an alphabetical scale.

It should be emphasized again that the use of weighted averages to quantify a freeway operation should not obviate the concept of balanced design discussed throughout this chapter. Each critical location should be examined in relation to the selected level of service and every effort made to correct the design at substandard points, so that operating conditions will not cause any element to function appreciably below this level.

Example 9.8 in the typical problems which follow demonstrates the procedures for both the numerical and the approximate methods.

TYPICAL PROBLEM SOLUTIONS—FREEWAYS AND EXPRESSWAYS

EXAMPLE 9.1

Problem:
    Given:
        Rural 4-lane freeway.
        12-ft lanes.
        10-ft shoulder on right; 3-ft on left; and 36-ft wide median.
        Overall long section, in level terrain.
        Ideal alinement; average highway speed = 70 mph.
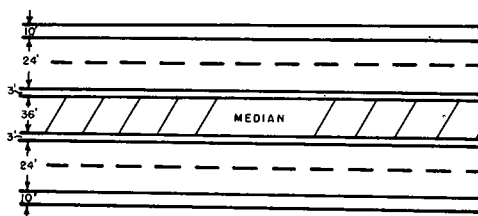
    5 percent trucks.
    1 percent intercity buses.
Determine:
    Service volumes for levels B and E (capacity).

Solution:

A review of the given conditions shows them to be largely ideal. No adjustment for lane width and lateral clearance is required; they meet requirements. However, even in this case, the tabulated values for ideal conditions in Table 9.1 cannot be used directly; adjustments for traffic factors (trucks and buses) must be considered.

For the long section under consideration, Table 9.3b applies. The few buses can be considered as passenger cars for this level terrain case.

Adjustment factor T_c for 5 percent trucks in level terrain = 0.95.

N, number of lanes in one direction = 2.

Capacity:

$C = 2,000$ N W $T_c = 2,000 \times 2 \times 1.00 \times 0.95 = 3,800$ vph, total for one direction.

Service Volume B (three methods demonstrated):

(a) $SV_B = 2,000$ N (v/c) W T_L (where v/c from Table 9.1 ≤ 0.50) = $2,000 \times 2 \times 0.50 \times 1.00 \times 0.95 = 1,900$ vph, total for one direction.

(b) $SV_B = c(v/c)(T_L/T_c)$ (where c is as computed above and T_L/T_c cancels out in this overall freeway case where T is identical for all levels) = $3,800 \times 0.50 = 1,900$ vph, total for one direction.

Caution: This method is applicable only where ideal geometrics exist, as is the case here.

(c) $SV_B = MSV$ W T_L (where MSV is from Table 9.1) = $2,000 \times 1.00 \times 0.95 = 1,900$ vph, total for one direction.

All methods produce the same result.

EXAMPLE 9.2

Problem:

Given:

Urban 6-lane freeway.

12-ft lanes.

10-ft shoulders on right, with obstructions at shoulder edge; 2-ft on left; and 5-ft median with barrier in center.

Individual grade, 3 percent, 1 mile long.

Alinement for 60-mph average highway speed.

5 percent trucks.

1 percent intercity buses.

PHF=0.91.
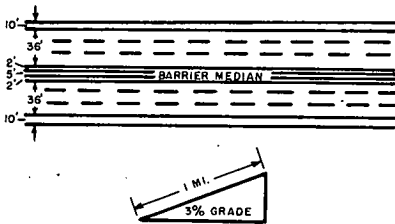
Determine: Service volumes for levels C and E (capacity).

Solution:

Capacity:

$N=3$.

W, for obstruction at 10 ft on right and approx. 4 ft on left (2-ft shoulder plus approx. 2 ft to median barrier), from Table 9.2 = 0.99.

(Basis: Obstruction 4 ft from pavement edge on one side only).

T_c, for 5 percent trucks on 3 percent grade 1 mi long, at capacity:

From Table 9.4, $E_T=8$

From Table 9.6, $T_c=0.74$

(The small volume of buses can be considered as passenger cars).

$c=2,000\ N\ W\ T_c=2,000\times3\times0.99\times 0.74=4,396$ vph, total for one direction.

Note: Where a commuter route is involved, and the barrier is continuous, the W factor might approach 1.00.

Service Volume C:

$N=3$.

$W=0.99$, as before.

T_L: From Table 9.4, $E_T=8$

From Table 9.6, $T_L=0.74$

v/c: Limiting value $\lesssim 0.80$ (PHF)

Practical working value $\lesssim 0.45$ (PHF).

Test of basic limiting SV (for demonstration purposes only; normally use working v/c ratio directly):

Limiting $SV_C=2,000\ N\ (v/c)\ W\ T_L= 2,000\times3\times(0.80\times0.91)\times0.99\times0.74=3,200$ vph, total for one direction.

(A check of Fig. 9.1 for the given v/c ratio $(0.80\times0.91=0.73)$, and AHS=60 mph, shows that level C operating speeds could not be attained at this volume under the given restricted average highway speed conditions.)

Application of working SV:

Working $SV_C=2,000\times3\times(0.45\times0.91)\times 0.99\times0.74=1,880$ vph, total for one direction, to provide level C operating speed of 50 mph.

EXAMPLE 9.3

Problem:

Given:

Rural 6-lane freeway.

12-ft lanes, adequate shoulders and clearances.

Individual grade, 4 percent, 1½ mi long.

Alinement for 70-mph average highway speed.

3 percent trucks.

Negligible buses.

PHF=0.77 in levels C and D.

Demand volume=2,700 vph, total in heavier direction, upgrade.
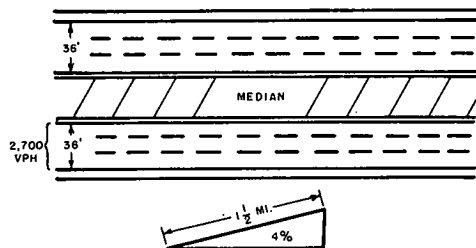
Determine: Level of service being pro-
vided on this upgrade.

Solution:
Inspection of Table 9.1, given 6-lane free-
way with PHF = 0.77, indicates that opera-
tion probably is in level B or C.
Assume level C for use in selecting adjust-
ments dependent on a known level.

$$\text{Base volume} = 2{,}000 \, N \, W \, T_L$$

where:
$N = 3$.
$W = 1.00$ (from Table 9.2 for ideal con-
ditions).
T_L: From Table 9.4, for level C, given
3 percent trucks on 4 percent 1½-
mi grade, $E_T = 12$.
From Table 9.6, for $E_T = 12$ and P_T
$= 3$, $T_L = 0.75$.
Base volume $= 2{,}000 \times 3 \times 1.00 \times 0.75$
$= 4{,}500$ vph.
v/c ratio $= 2{,}700/4{,}500 = 0.60$.
Inspection of Table 9.1 shows this value to
be slightly beyond level B limits. To check
level C, convert ratio, which here includes
influence of PHF, to basic form.
$0.60 = $ (Basic v/c ratio) (PHF)
Basic v/c ratio $= 0.60/$PHF $= 0.60/0.77$
$= 0.78$.
0.78 (PHF) < 0.80 (PHF); result is
within level C for 6-lane freeway.
Table 9.1 indicates that operating speed
for this ratio will be at or slightly above
55 mph, the limit for level C.
(Similarly, Figure 9.1, for v/c ratio of
0.60, indicates that operating speed of
55 mph will be attained.)
Assumption of level was correct; recom-
putation is not necessary.
Result: Level of service $= $ C.

EXAMPLE 9.4

Problem:
Given:
Rural 4-lane divided parkway of early
design.
10-ft lanes.
2-ft shoulders on right; none on left;
8-ft median.
Obstructions within 5 ft of pavement
edge on right; 2 ft on left.
Overall long section, in rolling ter-
rain.
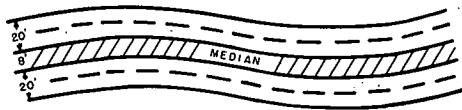
Restricted alinement; average high-
way speed $= 50$ mph.
No trucks or buses.
Determine: Service volumes for levels
B and E (capacity) and evaluate the re-
sults.

Solution:
Capacity:
$$c = 2{,}000 \, N \, W \, T_c$$
where:
$N = 2$.
W: From Table 9.2, for 10-ft lanes,
W for obstruction at 5 ft on both
sides $= 0.90$ (by interpolation
between 4 ft and 6 ft).
W for obstruction at 2 ft on both
sides $= 0.86$.
Use average factor, $W = 0.88$.
T_c: No trucks; no adjustment re-
quired.
$c = 2{,}000 \times 2 \times 0.88 \times 1.00 = 3{,}520$ vph.
Note: Length of section not significant
here, because no trucks are involved.
Service Volume B:
$$SV_B = c(v/c)(T_L/T_c)$$
T_L/T_c can be omitted because there
are no trucks.
v/c, in Table 9.1, for level B and
AHS $= 50$ mph, is found to be a
blank.

Conclusion: This parkway, with its re-
stricted alinement, is not capable of pro-
viding level B service. In fact, inspection
of Table 9.1 shows that it could not pro-
vide level C service. In other words, it
cannot provide high-speed operation, re-
gardless of volumes.

EXAMPLE 9.5

Problem:
Given:
Urban 4-lane expressway.
11-ft lanes.
Curbed, with 6-in. curbs at pavement
edge; 12-ft median.

Obstructions 2 ft from pavement edge on right; none on left.

Level 1-mile subsection under consideration; at-grade intersections involved, and little roadside friction.

8 percent trucks.

6 percent intercity buses.

PHF=0.91.

Determine: Service volumes for levels D and E (capacity).

Solution:

Capacity:

$$c = 2,000 \, N \, W \, T_c$$

where:

$N = 2$.

W: From Table 9.2, for 11-ft lanes with obstructions on one side only at 2 ft, $W = 0.94$. (Note: Curb is not high enough to be a lateral obstruction).

T_c: From Table 9.4, for 8 percent trucks on 1-mi subsection, $E_T = 2$.

From Table 9.6, for $E_T = 2$ and $P_T = 8$, $T_c = 0.93$.

B_c: Bus volumes are relatively high, so are considered separately.

From Table 9.5, $E_B = 1.6$.

From Table 9.6, for $E_B = 1.6$ and $P_B = 6$, $B_c = 0.96$ (by interpolation).

$c = 2,000 \times 2 \times 0.94 \times 0.93 \times 0.96$
$= 3,360$ vph.

Service Volume D:

All of above adjustments are the same at level D as at capacity.

$$SV_D = c(v/c)(T_L/T_c)$$

T_L/T_c term can be omitted because adjustments are the same.

v/c, from Table 9.1, $= 0.90$ (PHF).

$v/c = 0.90 \times 0.91 = 0.82$

$SV_D = 3,360(0.82) = 2,755$ vph.

EXAMPLE 9.6

Problem:

Given: Urban freeway with proposed design as in sketch.

Conditions:

Fully adequate freeway geometrics, including 12-ft lanes, clearances well over 6 ft, level terrain, ideal through alinement for 70-mph AHS, and well-designed ramp junctions.

Demand volumes as shown in sketch. Trucks negligible except as shown in sketch.

Buses negligible.

PHF=0.91.

Each line in sketch represents a proposed traffic lane.

Level of service C desired.

Determine:

(a) Adequacy of proposed design, in relation to desired level of service C with given peak-hour factor.

(b) Desirable revisions of design.

Solution:

(a) Adequacy of design for level C:

Freeway subsection 0-1:

Given conditions are ideal; service volume can be read directly from Table 9.1.

$SV_C = 2,750$ pcph, from Table 9.1.

No conversion for trucks necessary; trucks negligible. Then, $SV_C = 2,750$ vph.

Given $V = 2,100$ vph, with negligible trucks.

$2,100 < 2,750$; satisfactory.

Ramp 1:

Figure 8.2 applies; negligible trucks.

$V_1 = 136 + 0.345 \, V_f - 0.115 \, V_r = 136 + (0.345 \times 2,100) - (0.115 \times 600) = 792$ vph.

Merge $=792+600=1,392$ vph.
    $1,392 < 1,550$, from Table 8.1; sat-
    isfactory.
Freeway subsection 1-2:
    As before, from Table 9.1, for given
    ideal conditions including negli-
    gible trucks,
    $SV_C = 2,750$ vph.
    Given $V = 2,100 + 600 = 2,700$ vph.
        $2,700 < 2,750$; satisfactory.

Ramp 2:
    Figure 8.3 applies; negligible trucks.
    $V_1 = 165 + 0.345 \ V_t + 0.520 \ V_r = 165$
        $+ (0.345 \times 2,700) + (0.520 \times 300) =$
        $1,253$ vph.
    Diverge $= 1,253$ vph.
        $1,253 < 1,650$, from Table 8.1;
        satisfactory.

Freeway subsection 2-3:
    SV_C remains 2,750 vph, given negli-
    gible trucks.
    Demand $V = 2,700 - 300 = 2,400$ vph
    $2,400 < 2,750$; satisfactory.

Ramp 3:
    Figure 8.2 applies; negligible trucks.
    $V_1 = 136 + 0.345 \ V_f - 0.115 \ V_r = 136$
        $+ (0.345 \times 2,400) - (0.115 \times 700) =$
        884 vph.
    Merge $= 884 + 700 = 1,584$ vph.
        $1,584 > 1,550$, from Table 8.1; not
        satisfactory for level C.

Freeway subsection 3-4:
    SV_C remains 2,750 vph, given negli-
    gible trucks.
    Demand $V = 2,400 + 700 = 3,100$ vph.
    $3,100 > 2,750$; not satisfactory.

Ramp 4:
    Figure 8.4 applies; negligible trucks.
    $V_1 = 202 + 0.362 V_t + 0.496 V_r -$
    $0.069 \ D_u + 0.096 \ V_u = 202 + (0.362$
    $\times 3,110) + (0.496 \times 600) - (0.069$
    $\times 1,100) + (0.096 \times 700) = 1,613$
    vph, diverge.
    $1,613 < 1,650$, from Table 8.1; satis-
    factory but borderline.

Freeway subsection 4-5:
    SV_C remains 2,750 vph, given negli-
    gible trucks.

Demand $V = 3,100 - 600 = 2,500$ vph.
    $2,500 < 2,750$; satisfactory.
Ramp 5:
    Figure 8.2 applies.
    $V_1 = 136 + 0.345 \ V_f - 0.115 \ V_r = 136$
        $+ (0.345 \times 2,500) - (0.115 \times 400) =$
        953 vph.
    Merge $= 953 + 400 = 1,353$ vph.
        $1,353 < 1,550$, from Table 8.1; sat-
        isfactory.

Note: 4 percent trucks is under the
    5 percent limit for trucks for this
    procedure, so adjustment of final
    result for trucks is not necessary.

Freeway subsection 5-6:
    SV_C remains 2,750 pcph, but trucks
    must here be considered.
    Demand $V = 2,500 + 400 = 2,900$ vph.
    4 percent trucks in 400-vph flow $=$
    16 trucks.
    $16/2,900 = 0.006$; say 1 percent
    trucks.
    From Table 9.4 for level terrain, E_T
    $= 2$.
    From Table 9.6 for $E_T = 2$ and 1 per-
    cent trucks, $T_L = 0.99$.
    $SV_C = 2,750 \times 0.99 = 2,723$ vph.
    $2,900 > 2,723$; not satisfactory.

Weaving section 6-7:
    Figure 7.4 applies.
    Length (using graphical chart):
        V_{wc_1}: From Table 9.4, $E_T = 2$.
            From Table 9.6, for $E_T = 2$
            and 6 percent trucks, $T_L =$
            0.94. $V_{we_1} = 800/0.94 = 851$
            pcph.
        V_{wc_2}: No adjustment necessary; no
            trucks.
        $V_{wc_1} + V_{wc_2} = 851 + 700 = 1,551$
            pcph.
        For $V_{wc_1} + V_{wc_2} = 1,551$ pcph and
            $L = 1,200$ ft, using chart, quality
            of flow $= III$ with $k = 3.0$ (used
            to nearest tenth).
        From Table 7.3, for freeways, this
            is acceptable, though minimum,
            level C.

    Width (using formula):
        Average SV_C on approaches:

Average percent trucks

$1,700 \times 0.03 = 51$
$2,900 \times 0.01 = 29$
$1,600 \times 0.00 = 0$
$2,200 \times 0.01 = 22$ ⎱
$800 \times 0.06 = 48$ ⎰

$\overline{9,200} \qquad \overline{150}$

$150/9,200 = 0.0162 \simeq 2$ percent trucks.

For $E_T = 2$ and 2 percent trucks, $T_L = 0.98$.

Average $SV_C = 2,750 \times 0.98 = 2,695$ vph for 2 lanes.

$2,695/2 = 1,348$ vph per lane.

$N = [V + (k-1)(V_{w_2})]/SV_C = [1,700 + 2,900 + (3.0 - 1.0)(700)]/1,348 = 4.5$ lanes.

The proposed 3 lanes are not adequate.

Freeway subsection 7-8:

SV_C remains 2,750 pcph, and no trucks are in this subsection, so this again becomes equivalent to 2,750 vph.

Demand $V = 1,600$ vph.

$1,600 < 2,750$; satisfactory.

Note: The criteria employed should assure that the operating speed criteria have been met, if volume requirements are met, in all subsection checks above, but Figure 9.1 can be used for a final check.

(b) Desirable revisions of design:

A review of the findings in Part (a) indicates satisfactory conditions with the following exceptions:

1. Ramp 3 merge is excessive.
2. Freeway subsection 3-4 is inadequate.
3. Ramp 4 diverge is borderline.
4. Freeway subsection 5-6 is inadequate.
5. Weaving section 6-7 is of inadequate width.

Point 3 to Point 4:

It appears that an auxiliary lane between ramp junctions 3 and 4 would be desirable; it probably would correct deficiencies 1, 2, and 3, above. Test this proposal:

Ramp 3:

Figure 8.7 applies, with related use of Figure 8.20.

$V_1 = 281 + 0.400 \, V_f - 0.225 \, D_d + 0.394 \, V_d = 281 + (0.400 \times 2,400) - (0.225 \times 1,100) + (0.394 \times 600) = 1,230$ vph.

Lane 1 through volume $= V_1 -$ Ramp 4 volume $= 1,230 - 600 = 630$.

Check at midpoint of auxiliary lane using Figure 8.20.

Lane 1 volume = Through volume + Vehicles entered + Vehicles not yet exited $= 630 + (0.57 \times 700) + (0.25 \times 600) = 1,179$ vph.

$1,179 < 1,550$, from Table 8.1; satisfactory.

Auxiliary lane volume:

V_1 (upstream) $+ V_r$ (on) $- V_1$ (midpoint) $= 1,230 + 700 - 1,179 = 751$ vph, obviously satisfactory.

Weaving in 500 ft: Obviously satisfactory.

Across-all-freeway-lanes check, in subsection 3-4:

V (upstream) $+ V_r$ (on) $- V$ (auxil. lane) $= 2,400 + 700 - 751 = 2,349$ vph.

$2,349 < 2,750$; satisfactory.

Ramp 4: Previous check of ramp 3 has shown that operation approaching ramp 4 will now be fully acceptable.

Result: Auxiliary lane 3-4 corrects deficiencies 1, 2, and 3.

Subsection 5-6:

Simple case of inadequate width. Revise to 3 lanes.

$SV_C = 4,350 \times 0.99 = 4,310$.
$2,900 < 4,310$; satisfactory.

Weaving section 6-7:

Requires additional lanes. Revise to 5 lanes.

Conclusions:

Revisions necessary—

1. Add auxiliary lane, on-ramp 3 to off-ramp 4.

2. Widen subsection 5-6 to three lanes each direction.
3. Widen weaving section 6-7 to five lanes.

EXAMPLE 9.7

Problem:

Given: Rural expressway (west-to-east flow under consideration, see sketch).
4-lane divided expressway, with wide median.
11-ft through lanes.
Lateral clearances adequate.
No significant grades.
Good alinement.
Little roadside friction.
Each line on sketch indicates a lane.
Average rural conditions.
Demand volumes: As shown on sketch.
Intersection 1:
  Minor crossroad.
  Total cycle time = 90 sec.
  Expressway green time = 60 sec.
  Right turns = 3%.
  Left turns = 2% (These do not cross opposing traffic; storage is available after turning to wait for cross-road green.)
Intersection 2:
  Major crossroad.
  Total cycle time = 90 sec.
  Expressway green time:
    Through and right = 33 sec.
    Left (separate) = 15 sec.
    (Right turn does not move during left time).
  Right turns = 15%.
  Left turns = 12%, using separate added turn lane 10 ft wide.
Intersection 3 (widened to 3 11-ft lanes through intersection):
  Crossroad minor, but widens to 4 lanes so requires only short green time.
  Total cycle time = 60 sec.
  Expressway green time = 42 sec, simultaneous for all movements.
  Right turns = 4%.
  Left turns = 3% (Again, these do not cross opposing traffic; storage is available after turning to wait for crossroad green).
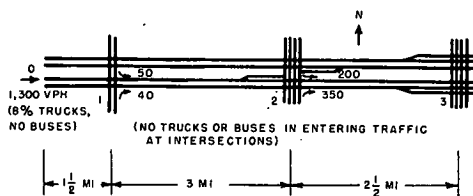
Determine: Service volume B of each element and relationship of demand volume to that service volume. Also, evaluate results.

Solution:

No. of passenger cars and trucks entering at point 0:
  $1,300 \times 0.08 = 104$ trucks.
  $1,300 - 104 = 1,196$ passenger cars.

Expressway subsection 0-1:
  $SV_B = 2,000\, N\, (v/c)\, W\, T_L$
  where
    $N = 2$.
    $v/c = 0.50$, from Table 9.1.
    $W = 0.97$, from Table 9.2.
    T_L: From Table 9.4, $E_T = 2$.
      From Table 9.6, $T_L = 0.93$.
    $SV_B = 2,000 \times 2 \times 0.50 \times 0.97 \times 0.93$
      $= 1,805$ vph.
  Demand volume = 1,300 vph.
    $1,300 < 1,805$; satisfactory.

Intersection 1:
  Figure 6.10 applies.
    Approach width = 22 ft.
    L.F. for intersection level B = 0.1.
    Chart volume = 1,500 vphg.
    Adjustments:
      G/C ratio = 60/90 = 0.67.
      Right turns, 3% (Table 6.4) = 1.035.
      Left turns, 2% (Table 6.4, since influence is like that on one-way) = 1.04.
      Trucks, 8% (Table 6.6) = 0.97.
    $SV_B = 1,500 \times 0.67 \times 1.035 \times 1.04 \times 0.97 = 1,050$ vph.
      $1,300 > 1,050$; not satisfactory.

Expressway subsection 1-2:
  Demand volume = $1,300 + 50 + 40 = 1,390$ vph.
    $104/1,390 = 0.075$; say 7% trucks.

No change in factors.

   SV_B remains 1,805 vph.

   1,390 < 1,805; satisfactory.

Intersection 2:

  Figure 6.10 applies.

  Through and right movements:

    Approach width for through and right = 22 ft.

    L.F. = 0.1.

    Chart volume = 1,500 vphg.

    Adjustments:

     G/C ratio = 33/90 = 0.37

     Right turns, 15% (Table 6.4) = 0.975.

     Left turns, 0% for this step (Table 6.4) = 1.10.

     Trucks, 8% = 0.97.

     1,500 × 0.37 × 0.975 × 1.050 × 0.97 = 551 vph, through and right.

  Left turns:

    G/C ratio = 15/90 = 0.17.

    8% trucks (Table 6.6) = 0.97.

    For level B, and no trucks, 800 × 0.17 × 0.97 = 132 vph, left.

  Overall service volume, assuming that through and right control:

    551/(1.00 − 0.12) = 627 vph, total vehicles arriving.

    With given demand distribution, left turns = 627 × 0.12 = 75.

    75 < 132; satisfactory left turns relative to through and right movements.

    1,390 > 627; not satisfactory for intersection level B.

Expressway subsection 2-3:

  Demand volume = 1,390 + 200 + 350 = 1,940 vph.

  104/1,940 = 5.3%, say 5% trucks.

  From Table 9.4, $E_T = 2$.

  From Table 9.6, $T_L = 0.93$.

  $SV_B = 2,000 × 2 × 0.50 × 0.97 × 0.93 = 1,805$ vph.

    1,940 > 1,805; not satisfactory.

Intersection 3:

  Figure 6.10 applies.

  Approach width = 33 ft.

  L.F. = 0.1.

  Chart value = 2,200 vphg.

Adjustments:

  G/C ratio = 42/60 = 0.70.

  Right turns, 4% (Table 6.4) = 1.015

  Left turns, 3% (Table 6.4, for one-way type influence) = 1.015

  Trucks, 5% (Table 6.6) = 1.00

  $SV_B = 2,200 × 0.70 × 1.015 × 1.015 × 1.00 = 1,587$ vph.

    1,940 > 1,587; not satisfactory.

Evaluation of results:

It is found that none of the intersections meet intersection level of service B criteria, and that freeway section 2-3 does not meet level B requirements.

It is important to remember, however, that intersection levels of service do not relate directly to uninterrupted flow levels; the criteria differ. It must be expected that a typical intersection, which interrupts smooth flow, will not provide service equal to the design level of the through highway unless the intersection is widened. It is essential, however, that its *capacity* at least equal the design service volume.

At intersection 1, using Figure 6.10, the capacity is found to be (2,000/1,500) × 1,050 = 1,400 vph.

    1,300 < 1,400; satisfactory.

Intersection 1 will thus delay some traffic beyond one signal cycle, but will not produce a continuing back-up.

At intersection 2 the situation is more critical. Here, so much green time is required for the cross road, as well as for the separate turning movement, that the two through lanes cannot handle the through highway's design volume in the time available. Depending on the volume of opposing left-turn movements, a "leading green" for left turns might give some additional time for the W-to-E through flow. But with the heavy cross traffic, only major widening or grade separation can fully correct this location.

At intersection 3 conditions are reasonably good because the cross road has been widened sufficiently to accommodate its traffic in a short period of time, and the expressway
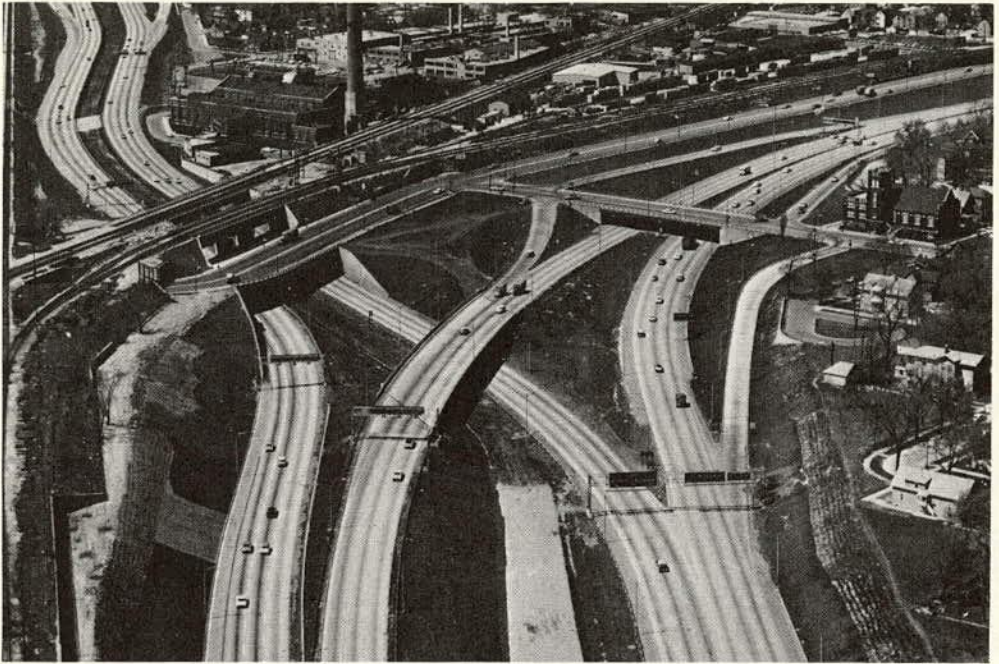
A major fork on a freeway network.

itself has been widened to make up for much of that lost time.

On expressway subsection 2-3 the heavy influx of traffic from the cross road has caused operation to fall somewhat into level C. Widening to 3 lanes will be required if level B is to be attained.

EXAMPLE 9.8

Part (a)

Problem: Determine the weighted level of service for the 4-lane freeway section shown, which has already-determined level of service measures as shown (PHF= 0.77).
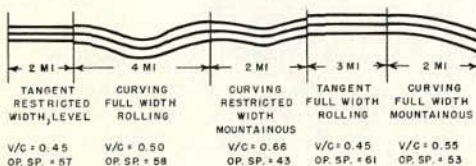
Solution:

v/c weighting:

$$0.45 \times 2 = 0.90$$
$$0.50 \times 4 = 2.00$$
$$0.66 \times 2 = 1.32$$
$$0.45 \times 3 = 1.35$$
$$0.55 \times 2 = \underline{1.10}$$
$$\overline{13} \quad \overline{6.67}$$

$6.67/13 = 0.51$, weighted v/c ratio; in level C.

Operating speed weighting:

$$57 \times 2 = 114$$
$$58 \times 4 = 232$$
$$43 \times 2 = 86$$
$$61 \times 3 = 183$$
$$53 \times 2 = \underline{106}$$
$$\overline{13} \quad \overline{721}$$

$721/13 = 55.5$ mph, weighted operating speed; in level B.

Overall level of service is C, governed by v/c ratio.

(This is only slightly into level C,

however, as shown by v/c only 0.01 over the dividing line while operating speed is slightly better than limit of level B).

Part (b)

Problem: Determine the approximate overall level of service for the freeway and expressway section shown, which has a variety of elements without readily available numerical level of service measures. (PHF = 0.83, geometrics are ideal, and no trucks).

Solution:

	ON-RAMP	WEAVING SECTION			OFF-RAMP		ON-RAMP	AT-GRADE INTERSECTION		
JUNCTION POINTS	2,500'	3,500'	1,500'	5,000'		7,000'		4,000'	1,500'		
	500'	2,500'	500'	500'		500'		500'			
INFLUENCE AREA	2,000'	3,000'	500'	2,500'	2,000'	3,000'	6,000'	3,000'	1,500'	1,500'	
SITUATION:	THRU	MERGE	THRU	WEAVE**	THRU	DIVERGE	THRU	MERGE	THRU	THRU	
GIVEN DEMAND VOLUME (VPH)*	1,900	1,100	2,400	1,300	2,100	1,400	1,500	1,200	1,900	1,800	
	B / 2,000	B / 1,200	C / 2,500	C / 2,000	C / 2,500	C / 1,500	B / 2,000	B / 1,200 (EXACT)	B / 2,000	E / 2,000	B / 2,000
NEAREST SERVICE VOLUME LIMITS*	1,400 A	1,000 A	2,000 B	1,100 B	2,000 B	1,300 B	1,400 A		1,400 A	1,800 D	1,400 A

LEVEL OF SERVICE A–E chart

*DEMAND VOLUMES AND SURROUNDING SERVICE VOLUME LIMITS ARE DIRECTLY AVAILABLE FROM SUBSECTION-BY-SUBSECTION ANALYSIS OF ELEMENTS.

**WEAVE LIMITS TAKEN FROM CHAP. SEVEN FOR MAJOR WEAVES; IN FIGURE 7.4, NUMBER OF WEAVING VEHICLES RATHER THAN NUMBER OF LANES FOUND TO CONTROL.

Conclusion: Average overall level of service is approximately the limit of **B**, with the exception of the at-grade intersection, which is very close to capacity. This point restriction is so far "out of line" with the remaining operation that it should be reported separately, rather than being averaged.

STREETS AND HIGHWAYS WITHOUT ACCESS CONTROL

The previous chapter has discussed traffic operations on freeways and other expressways, or highway facilities designed for the primary purpose of the rapid movement of traffic. Relatively isolated from outside influences except at specific locations, the internal traffic stream characteristics are the major determinants of the traffic service provided.

However, the vast majority of rural highways and city streets do not fall into this category. On most highways, the initial purpose of swift, efficient movement is compromised by provision of direct access to and from adjacent roads, streets, and abutting properties. Sometimes the access is essential; in other cases, gaining access control simply is not economically feasible. The typical street or highway thus fulfills a multiple function of traffic service and land use service, and, in the process, the description of traffic operations becomes more complex. In contrast with typical uninterrupted flow experiencing only occasional potential conflict areas (as on expressways), traffic flow on the majority of streets and highways can be thought of as an almost continual succession of potential conflicts from internal and external influences, whose effects produce somewhat different analysis criteria from those presented in the previous chapter. This chapter describes these differing procedures for streets and highways without access control.

Because of the widely varying operating characteristics and factors which may apply, this chapter gives separate consideration to five different highway types. First, multilane highways which are without access control and/or undivided are considered. Second, 2-lane highways are covered, followed by (third) brief mention of 3-lane roads. All of these sections pertain primarily to rural conditions, where the number of intersections and access points is few to moderate. Fourth, urban arterial streets are covered separately, inasmuch as the influence of urban traffic regulations, controls, and frequent access points results in a considerably different type of operation. Finally, major streets in the central business district are discussed.

GENERAL CONSIDERATION OF LEVELS OF SERVICE AND SERVICE VOLUMES

For the remaining types of highways, levels of service differ from those for freeways and expressways; further, they differ from each other, depending on the particular type of highway involved. For rural multilane highways, they are very similar to those for freeways; and for rural 2-lane highways they are reasonably similar. However, values for urban arterials are quite different, as is the concept itself, and at the other extreme (downtown streets) they are very different and not yet well defined.

Thus, absolute measures of level of service are meaningful only within a given highway category, because by definition each category has its own range of levels of service, including its own "best" level of service. Conditions at this "best" level of service to a large degree are controlled by external influences rather than internal traffic stream characteristics. Level of service must, then, apply to a section of roadway of a given type, representing the average effect of operating levels at each point along that section. Differing types should not be combined for analysis; for instance, a highway which is partly 4-lane and partly 2-lane

should be divided into and reported as two separate sections, as a minimum.

Evaluation of the interacting effects of level of service and maximum service volume on highways other than expressways involves many factors. External influences have more effect on traffic flow on ordinary highways than on freeways at all volume levels because of their proximity and increased frequency. This effect is reflected primarily in reduced speeds as compared to freeways, at any given volume. These speed changes thus differ in their effect from those resulting from a speed limit intentionally applied for safety or other reasons. Such posted speed limits, when lower than those shown attainable by the appropriate speed-volume curves, can be maintained with little change until the volume shown for that speed is reached.

In this chapter, as in Chapter Nine, speed is used, together with v/c ratio, in defining levels of service. As before, a series of speed values are designated as limits of the several levels of service. However, two forms of speed measurement are used, operating speed and average overall travel speed, the particular one employed for any given highway type depending on the research studies used as references and the particular significance that each has for the type of roadway under consideration. *Operating speed,* as before, represents the maximum safe speed for given traffic conditions that an individual vehicle can travel if the driver so desires, without exceeding the design speed at any point. *Average overall travel speed* is the average of individual vehicular speeds over a length of highway, and represents what all vehicles, acting as a group, can be expected to do. It can be obtained from the average travel times of a group of vehicles passing through the section.

When dealing with essentially uninterrupted flow conditions, normally associated with rural highways, operating speed will normally be the speed criterion. As before, it indicates the highest feasible speeds at a given volume, and eliminates the variability in observed speeds caused by individual driver desires. A given highway or street will have a free-flow operating speed, or a maximum safe speed at extremely low volumes,

limited by: (1) the physical characteristics of the roadway, and (2) the frequency and duration of the infrequent fixed traffic interruptions (Speed limits may legally prevent attainment of low-volume operating speeds, as defined.) Operating speeds at any given level of service, then, reflect the influence of the volume at that level. The discussions of multilane and 2-lane highways, covering basically rural conditions, therefore normally relate volumes, as represented by the v/c ratio, to operating speeds.

As in the freeway case discussed in Chapter Nine, the basic v/c ratio limits here defining the several levels of service on multilane and 2-lane highways with uninterrupted flow, from a volume standpoint, apply to ideal alinement, or 70-mph average highway speeds. Again, as in the freeway case, these limits are generally unrealistic when applied to highways of lower design standards, because they indicate volumes higher than could be attained at the operating speed value also defining the particular level of service. Working v/c ratios more appropriate for lower average highway speeds are therefore included in the procedures that follow. These are approximate, in the case of multilane highways. However, in the 2-lane case, where they are an important, frequently used consideration, they are based on rather detailed studies.

On urban streets in well-developed areas, on the other hand, there may be frequent traffic interruptions, and speed limits may relate more to safety to the general public than to optimum traffic flow. On such streets and on other highways having frequent traffic interruptions or arbitrary speed limits for safety, average overall travel speed is a more suitable criterion. The range between possible and actual average speeds diminishes rapidly, with lower speed limits and increased frequency of. traffic interruptions. Alinement is seldom a significant consideration except in extreme cases. Traffic characteristics tend to become based on group, rather than individual, vehicle operations. When considering (1) city streets, (2) roadways with frequent traffic signalization or stop controls, or (3) routes with relatively low speed limits in relation to geometric conditions, it is recommended that analysis be made by the methods given later in this

chapter, for urban arterials and downtown streets, on the basis of average overall travel speeds. Here, only approximate v/c ratio criteria can be established, inasmuch as urban highway capacities are widely variable.

In this chapter, it will be noted that no specific application of peak-hour factors is made to uninterrupted flows. Little research has been conducted regarding peaking on ordinary rural highways; this may be be- cause the restrictive effects of the relatively large number of frictional elements has been considered to mask or damp the effects of peaking. Where, in a specific problem, an ordinary rural highway is encountered which approaches freeway characteristics or on which peaking is evident, judgment should be exercised in the possible adaptation of the peak-hour factors presented in Chapter Nine to the case at hand.

MULTILANE RURAL HIGHWAYS

This section covers multilane highways that cannot be classified as freeways or expressways because they are undivided, or because they lack significant control of access features, or both. Although freeways and expressways are also multilaned, with few exceptions, their unique characteristics with regard to access control and design features warrant their separate consideration in Chapter Nine.

On ordinary multilane highways in rural areas, just as on freeways, relatively uninterrupted flow is usually found, even though a variety of interferences exist which may adversely affect flow as compared to freeway flow. However, because of the many different types of conditions that may be found on particular highways, uninterrupted flow cannot always be maintained. Therefore, in this section level of service is first discussed for uninterrupted flow conditions, following which the effects of fixed traffic interruptions or restrictive conditions are considered.

An important difference between ordinary multilane and freeway operation, as described in this manual, is that the presence of more than two lanes in one direction on an uncontrolled multilane highway does not necessarily produce the same predictable increase in efficiency that has been shown to occur at intermediate volume levels on freeways and expressways. Often, the inner lanes are as adversely influenced by medial friction and by turns to and from the left side of the road as the outer lanes are by slow traffic and right-side frictions. For the purposes of analysis, therefore, ordinary multilane highways are considered to carry the same service volumes per lane in one direc- tion, regardless of the number of lanes in that direction. Operating speeds reflect the average conditions in all lanes.

Even under very light demand, traffic volumes have a measurable effect on operating speeds. Unlike the freeway case, vehicles entering, leaving and crossing the main roadway at a multitude of points, as well as those traveling in the opposing traffic flow, are present and may have a significant adverse effect on traffic operations. Increased traffic volumes produce a corresponding reduction in operating speeds.

Ideal conditions for ordinary multilane highways without access control are much the same as for freeways, including 12-ft lanes, fully adequate lateral clearances and shoulders, alinement for 70-mph average highway speed, and no commercial vehicles. Their average per lane capacities, under ideal conditions, are also the same—2,000 passenger vehicles per hour. However, the likelihood of ideal conditions is considerably less; hence, in many cases there must be substantial application of the adjustment factors discussed in Chapter Five and included in this section.

In Chapter Three, typical speed distributions and speed-volume relationships have been shown for ordinary multilane highways under ideal conditions as Figure 3.24 and Figures 3.36 and 3.39, respectively. Because, as just mentioned, relatively few ordinary multilane highways have ideal prevailing conditions, these curves would seldom represent an actual highway's performance correctly; they should not be used for computational purposes.

LEVELS OF SERVICE

For ordinary multilane highways under uninterrupted-flow conditions, the characteristics of the several levels of service are much the same as those for freeways, presented in Chapter Nine. The main differences are somewhat lower operating speeds at most volume levels, which result in slightly different operating speed and v/c ratio limits for certain of the levels.

Described briefly, starting from the zero-volume condition, free flow, level A extends as before to the point where the operating speeds are decreased not more than 10 mph below free-flow operating speeds at very low volumes; here, volumes will not exceed 30 percent of capacity. Its limit, under ideal conditions, is 600 passenger cars per lane per hour at an operating speed of 60 mph. In this level, average speeds are likely to be influenced by speed limits. Level B, marking the beginning of the stable-flow area, represents the volume at which most of the drivers are traveling at headways where the actions of the preceding vehicle have some influence on them; it will not exceed 50 percent of capacity. This is 1,000 passenger cars per lane per hour, at a 55-mph operating speed, under ideal conditions. Level C represents a continuation of stable flow to a volume not exceeding 75 percent of capacity, or 1,500 passenger cars per lane per hour under ideal conditions, maintaining at least a 45-mph operating speed. Level D approaches unstable flow, at volumes up to 90 percent of capacity, but accommodates 1,800 passenger cars per lane per hour at an operating speed of about 35 mph under ideal conditions. Level E, of course, represents capacity, or 2,000 passenger cars per lane per hour under ideal conditions, with operating speeds of about 30 mph, somewhat under one-half of free-flow operating speed. Finally, level F is a forced-flow, congested condition with widely varying volume characteristics and operating speed capabilities falling below 30 mph. Often, on ordinary multilane highways, level F is reached directly from level D, as traffic demand rises and volumes increase, the extremely unstable level E being bypassed entirely.

Although, as previously mentioned, peak-

An ordinary multilane highway without access control.

hour factors are not directly considered on ordinary highways, it may be assumed that the maximum service volumes given for levels C and D approximately represent the maximum volumes that can be sustained for long (level C) or short (level D) periods of time without undue restriction, delay or likelihood of breakdown of flow.

Table 10.1 summarizes these relationships between levels of service, operating speeds, and v/c ratios on ordinary multilane highways (as normally found in rural areas), both for ideal and restricted alinement, and also shows maximum service volumes and capacity under ideal conditions.

CRITICAL ELEMENTS REQUIRING CONSIDERATION

An ordinary multilane highway with ideal conditions (12-ft lanes, adequate shoulders, no commercial vehicles, and good alinement) is the exception rather than the rule. Normally, a typical highway will have, to a greater or lesser degree, restricting elements

TABLE 10.1—LEVELS OF SERVICE AND MAXIMUM SERVICE VOLUMES FOR MULTILANE HIGHWAYS, UNDIVIDED AND/OR WITHOUT ACCESS CONTROL, UNDER UNINTERRUPTED FLOW CONDITIONS (NORMALLY REPRESENTATIVE OF RURAL OPERATION)

LEVEL OF SERVICE	TRAFFIC FLOW CONDITIONS		SERVICE VOLUME/CAPACITY (v/c) RATIO			MAXIMUM SERVICE VOLUME UNDER IDEAL CONDITIONS, INCLUDING 70-MPH AHS (TOTAL PASSENGER CARS PER HOUR, ONE DIRECTION)		
	DESCRIPTION	OPERATING SPEED[a] (MPH)	BASIC LIMITING VALUE[a] FOR AHS OF 70 MPH	APPROXIMATE WORKING VALUE FOR RESTRICTED AHS OF		4-LANE HWY. (2 LANES ONE DIRECTION)	6-LANE HWY. (3 LANES ONE DIRECTION)	EACH ADDITIONAL LANE
				60 MPH	50 MPH			
A	Free flow	$\geqslant 60$	$\leqslant 0.30$	—[b]	—[b]	1200	1800	600
B	Stable flow (upper speed range)	$\geqslant 55$	$\leqslant 0.50$	$\leqslant 0.20$	—[b]	2000	3000	1000
C	Stable flow	$\geqslant 45$	$\leqslant 0.75$	$\leqslant 0.50$	$\leqslant 0.25$	3000	4500	1500
D	Approaching unstable flow	$\geqslant 35$	$\leqslant 0.90$	$\leqslant 0.85$	$\leqslant 0.70$	3600	5400	1800
E[c]	Unstable flow	30[d]	$\leqslant 1.00$			4000	6000	2000
F	Forced flow	<30[d]	Not Meaningful[e]			Widely variable (0 to capacity)		

[a] Operating speed and basic v/c ratio are independent measures of level of service; both limits must be satisfied in any determination of level.
[b] Operating speed required for this level is not attainable even at low volumes.
[c] Capacity.
[d] Approximately.
[e] Demand volume/capacity ratio may well exceed 1.00, indicating overloading.

which *must* be taken into account before its true ability to carry traffic can be determined. Hence, the maximum values given in Table 10.1 can seldom be used directly. Many typical highways are totally unable to provide service at level A, and level B may be unattainable on a significant number. In the computation procedures that follow, the adverse effects of restricted alinement are "built into" the criteria for restricted average highway speed conditions, whereas those of several other restrictive factors are handled by means of adjustment factors there presented.

In addition to those elements covered by specific adjustments, there are several other factors along ordinary multilane highways which may adversely affect their ability to offer uniform service throughout. These include rudimentary weaving areas, ramp junctions, at-grade intersections, business and private roadside development and associated access points, and a variety of other potential traffic interruptions.

The differences in effect of these various influences on ordinary multilane highways from those on freeways and expressways are primarily differences in scope and degree. On freeways, only occasional impediments are likely to be encountered, although the effect of a restriction may be noted for long distances if volumes are near the capacity of the restriction. On ordinary multilane highways impediments are likely to be found more frequently. In rural areas their effects may, again, extend for substantial distances. However, their zones of influence may be less in suburban areas because, with access uncontrolled, a restriction at one point may be more easily bypassed or avoided entirely by nearby local traffic, which continues to make effective use of the remainder of the highway.

Hence, on ordinary multilane highways having a substantial number of minor impediments, but only an occasional major one, it may sometimes prove unfeasible to quote an overall level of service including all adverse influences. Rather, development of a controlling level reflecting the minor restrictions, together with special analyses of the real influence of the apparent major re-

strictions in the light of local conditions, may prove a more useful approach.

Lane Width and Lateral Clearance

As discussed in Chapter Five, many ordinary multilane highways, particularly if relatively old, have lane widths of less than the ideal 12 ft, and lateral clearances from the edge of the traveled way to obstructions of less than the ideal 6 ft. Further, in most cases they are undivided; consequently, vehicles in the center lanes in each direction are restricted laterally by vehicles moving in the opposing direction. Table 10.2 presents adjustment factors which reflect the combined influence of these several factors on undivided multilane highways (Table 9.2 should be used for divided highways).

Generally, the significant lateral restrictions encountered on this class of highway will be more critical, and more abrupt in nature, than those few that are found on freeways. However, if any long continuous obstructions are present, the cautions discussed under this category in Chapter Nine should be considered. It should be noted, in Table 10.2, that the adjustments for "obstruction on right side only" already incorporate the effect of opposing traffic; no further adjustment is needed. Therefore, the adjustments for "obstruction on both sides" should be used only where some physical obstruction encroaches on the center of the roadway, closer than would the opposing flow of traffic; such obstructions would include centerline barriers, bridge structural elements, piers, and similar encroachments.

Trucks, Buses, and Grades

The interrelated effects of trucks, buses, and grades on service volumes and capacities of highways of all types have been discussed in Chapter Five, and the limited knowledge of these effects on service volumes and the capacity of multilane highways has been applied in detail to freeways and expressways in Chapter Nine. Although minor differences probably exist in actuality between the specific effects on freeways and those on ordinary multilane highways, available research results do not yet justify such refinement. Therefore, the discussion and adjustments presented in that chapter can be

considered to be equally applicable to ordinary multilane highways, except in those special cases where a traffic-interrupting feature exists on the grade; reference should be made to that section for general discussion.

For computational convenience, Tables 10.3 through 10.6 are included here; they contain the same truck and bus adjustment factors for ordinary multilane highways as do Tables 9.3 through 9.6, respectively, for freeways.

Weaving Areas

Chapter Seven covers major weaving, which is most commonly associated with freeways, and Chapter Eight considers "one-sided" weaving. There are many instances, however, where a weaving section is used in

TABLE 10.2—COMBINED EFFECT OF LANE WIDTH AND RESTRICTED LATERAL CLEARANCE ON CAPACITY AND SERVICE VOLUME OF UNDIVIDED MULTILANE HIGHWAYS WITH UNINTERRUPTED FLOW

DISTANCE FROM TRAFFIC LANE EDGE TO OBSTRUCTION (FT)	ADJUSTMENT FACTOR,[a] W, FOR LATERAL CLEARANCE AND LANE WIDTH							
	OBSTRUCTION ON RIGHT SIDE ONLY, OF ONE-DIRECTION TRAVELED WAY (INCLUDES ALLOWANCE FOR OPPOSING TRAFFIC ON LEFT)				OBSTRUCTIONS ON BOTH SIDES OF ONE-DIRECTION TRAVELED WAY[b,c]			
	12-FT LANES	11-FT LANES	10-FT LANES	9-FT LANES	12-FT LANES	11-FT LANES	10-FT LANES	9-FT LANES
(a) 4-LANE UNDIVIDED HIGHWAY, ONE DIRECTION OF TRAVEL								
6	1.00	0.95	0.89	0.77	N.A.	N.A.	N.A.	N.A.
4	0.98	0.94	0.88	0.76	N.A.	N.A.	N.A.	N.A.
2	0.95	0.92	0.86	0.75	0.94	0.91	0.86	N.A.
0	0.88	0.85	0.80	0.70	0.81	0.79	0.74	0.66
(b) 6-LANE UNDIVIDED HIGHWAY, ONE DIRECTION OF TRAVEL								
6	1.00	0.95	0.89	0.77	N.A.	N.A.	N.A.	N.A.
4	0.99	0.94	0.88	0.76	N.A.	N.A.	N.A.	N.A.
2	0.97	0.93	0.86	0.75	0.96	0.92	0.85	N.A.
0	0.94	0.90	0.83	0.72	0.91	0.87	0.81	0.70

(c) DIVIDED HIGHWAYS, ONE DIRECTION OF TRAVEL

Use adjustment factors from Table 9.2

[a] Same adjustments for capacity and all levels of service.
[b] Appropriate for use only where normally undivided roadway is temporarily separated into two roadways by obstructions such as centerline barrier, bridge structural elements, piers, and the like, which are closer than would be the opposing traffic.
[c] N.A. = Not applicable; use adjustment for obstruction on right side only. (In these cases, clearance is temporarily greater than the usual separation from opposing traffic, but adjustment for this temporary improvement is not feasible).

TABLE 10.3a—AVERAGE GENERALIZED PASSENGER CAR EQUIVALENTS OF TRUCKS AND BUSES ON ORDINARY MULTILANE HIGHWAYS, OVER EXTENDED SECTION LENGTHS
(INCLUDING UPGRADES, DOWNGRADES, AND LEVEL SUBSECTIONS)

LEVEL OF SERVICE		EQUIVALENT, E, FOR:		
		LEVEL TERRAIN	ROLLING TERRAIN	MOUNTAINOUS TERRAIN
A		Widely variable; one or more trucks have same total effect, causing other traffic to shift to other lanes. Use equivalent for remaining levels in problems.		
B through E	E_T, for trucks	2	4	8
	E_B, for buses[a]	1.6	3	5

[a] Separate consideration not warranted in most problems; use only where bus volumes are significant.

TABLE 10.3b—AVERAGE GENERALIZED ADJUSTMENT FACTORS FOR TRUCKS[b] ON ORDINARY MULTILANE HIGHWAYS, OVER EXTENDED SECTION LENGTHS

PERCENTAGE OF TRUCKS, P_T	FACTOR, T, FOR ALL LEVELS OF SERVICE		
	LEVEL TERRAIN	ROLLING TERRAIN	MOUNTAINOUS TERRAIN
1	0.99	0.97	0.93
2	0.98	0.94	0.88
3	0.97	0.92	0.83
4	0.96	0.89	0.78
5	0.95	0.87	0.74
6	0.94	0.85	0.70
7	0.93	0.83	0.67
8	0.93	0.81	0.64
9	0.92	0.79	0.61
10	0.91	0.77	0.59
12	0.89	0.74	0.54
14	0.88	0.70	0.51
16	0.86	0.68	0.47
18	0.85	0.65	0.44
20	0.83	0.63	0.42

[b] Not applicable to buses where they are given separate specific consideration; use instead Table 10.3a in conjunction with Table 10.6.

TABLE 10.4—PASSENGER CAR EQUIVALENTS OF TRUCKS ON ORDINARY MULTILANE HIGHWAYS, ON SPECIFIC INDIVIDUAL SUBSECTIONS OR GRADES

GRADE (%)	LENGTH OF GRADE (MI)	PASSENGER CAR EQUIVALENT, E_T									
		LEVELS OF SERVICE A THROUGH C FOR:					LEVELS OF SERVICE D AND E (CAPACITY) FOR:				
		3% TRUCKS	5% TRUCKS	10% TRUCKS	15% TRUCKS	20% TRUCKS	3% TRUCKS	5% TRUCKS	10% TRUCKS	15% TRUCKS	20% TRUCKS
0–1	All	2	2	2	2	2	2	2	2	2	2
2	¼–½	5	4	4	3	3	5	4	4	3	3
	¾–1	7	5	5	4	4	7	5	5	4	4
	1½–2	7	6	6	6	6	7	6	6	6	6
	3–4	7	7	8	8	8	7	7	8	8	8
3	¼	10	8	5	4	3	10	8	5	4	3
	½	10	8	5	4	4	10	8	5	4	4
	¾	10	8	6	5	5	10	8	5	4	5
	1	10	8	6	5	6	10	8	6	5	6
	1½	10	9	7	7	7	10	9	7	7	7
	2	10	9	8	8	8	10	9	8	8	8
	3	10	10	10	10	10	10	10	10	10	10
	4	10	10	11	11	11	10	10	11	11	11
4	¼	12	9	5	4	3	13	9	5	4	3
	½	12	9	5	5	5	13	9	5	5	5
	¾	12	9	7	7	7	13	9	7	7	7
	1	12	10	8	8	8	13	10	8	8	8
	1½	12	11	10	10	10	13	11	10	10	10
	2	12	11	11	11	11	13	12	11	11	11
	3	12	12	13	13	13	13	13	14	14	14
	4	12	13	15	15	14	13	14	16	16	15
5	¼	13	10	6	4	3	14	10	6	4	3
	½	13	11	7	7	7	14	11	7	7	7
	¾	13	11	9	8	8	14	11	9	8	8
	1	13	12	10	10	10	14	13	10	10	10
	1½	13	13	12	12	12	14	14	13	13	13
	2	13	14	14	14	14	14	15	15	15	15
	3	13	15	16	16	15	14	17	17	17	17
	4	15	17	19	19	17	16	19	22	21	19
6	¼	14	10	6	4	3	15	10	6	4	3
	½	14	11	8	8	8	15	11	8	8	8
	¾	14	12	10	10	10	15	12	10	10	10
	1	14	13	12	12	11	15	14	13	13	11
	1½	14	14	14	14	13	15	16	15	15	14
	2	14	15	16	16	15	15	18	18	18	16
	3	14	16	18	18	17	15	20	20	20	19
	4	19	19	20	20	20	20	23	23	23	23

TABLE 10.5—PASSENGER CAR EQUIVALENTS OF INTERCITY BUSES ON
ORDINARY MULTILANE HIGHWAYS, ON SPECIFIC INDIVIDUAL
SUBSECTIONS OR GRADES

GRADE[a] (%)	PASSENGER CAR EQUIVALENT,[b] E_B	
	LEVELS OF SERVICE A THROUGH C	LEVELS OF SERVICE D AND E (CAPACITY)
0–4	1.6	1.6
5[c]	4	2
6[c]	7	4
7[c]	12	10

a All lengths.
b For all percentages of buses.
c Use generally restricted to grades over 1/2 mile long.

connection with highways having no access control or where weaving occurs even if not by design. These range from maneuvers of traffic entering from a cross street on one side of a flow and leaving via another cross street on the other side in a short distance, through rotaries of low design, to fully adequate weaving sections. In the latter case, the weaving area of the roadway takes on the characteristics of an access-controlled roadway in that little or no side friction is normally present. Therefore, such weaving area capacity computations can be accomplished as if the section truly had controlled access, in accordance with the procedures established in Chapter Seven and further applied in Chapter Nine. However, in the remaining cases the influence of a variety of frictional elements must be considered.

Where sections have become encumbered with traffic signals in the section or on the approach legs, usually because the sections had insufficient capacity to handle the uncontrolled demand, those signalized intersections normally govern. They are analyzed by means of the procedures in Chapter Six. In the more difficult cases, however, even signalization may be insufficient to control the section, and the section's inherent char-

acteristics may still govern the very limited capacity. Such capacities cannot be computed by means of procedures in this manual; they must be determined through local study.

Ramp Entrances or Exits

Chapter Eight discusses capacity determination for ramps, but the procedures there described apply principally to a high-type ramp terminal linking the ramp turning roadway with a controlled-access freeway. In some cases, both ends of a ramp turning roadway may be junctions with freeways, but more often, only one terminal is at a freeway, the other being a link to a non-controlled- or partially-controlled-access highway. Any such lower-standard junction should be analyzed as a connection to an expressway, or to an ordinary highway or street, as the case may be.

CLOVERLEAF AND DIRECT CONNECTION
  INTERCHANGES

The ramps on these types of interchanges generally require the same type of freeway ramp junction capacity analysis at both termini, regardless of the fact that one roadway may be a freeway and the other a non-

controlled-access facility. This is true because, through the interchange, the non-controlled-access roadway takes on the same controlled-access characteristics as the controlled roadway. Therefore, Chapters Seven and Eight, as they apply to diverging and merging traffic in cloverleaf interchanges, are applicable. In practice, if the interchange is relatively close to signalized intersections traffic past the ramp junction will be in platoons to a greater degree than on a freeway; hence, ramp operation may be somewhat more on a queue-and-clear basis than would be true at a freeway junction.

TABLE 10.6—ADJUSTMENT FACTORS[a] FOR TRUCKS AND BUSES ON INDIVIDUAL HIGHWAY SUBSECTIONS OR GRADES ON ORDINARY MULTILANE HIGHWAYS (INCORPORATING PASSENGER CAR EQUIVALENT AND PERCENTAGE OF TRUCKS OR BUSES)[b]

PASSENGER CAR EQUIVALENT, E_T OR E_B[c]	TRUCK ADJUSTMENT FACTOR T_c OR T_L (B_c OR B_L FOR BUSES)[d]														
	PERCENTAGE OF TRUCKS, P_T (OR OF BUSES, P_B) OF:														
	1	2	3	4	5	6	7	8	9	10	12	14	16	18	20
2	0.99	0.98	0.97	0.96	0.95	0.94	0.93	0.93	0.92	0.91	0.89	0.88	0.86	0.85	0.83
3	0.98	0.96	0.94	0.93	0.91	0.89	0.88	0.86	0.85	0.83	0.81	0.78	0.76	0.74	0.71
4	0.97	0.94	0.92	0.89	0.87	0.85	0.83	0.81	0.79	0.77	0.74	0.70	0.68	0.65	0.63
5	0.96	0.93	0.89	0.86	0.83	0.81	0.78	0.76	0.74	0.71	0.68	0.64	0.61	0.58	0.56
6	0.95	0.91	0.87	0.83	0.80	0.77	0.74	0.71	0.69	0.67	0.63	0.59	0.56	0.53	0.50
7	0.94	0.89	0.85	0.81	0.77	0.74	0.70	0.68	0.65	0.63	0.58	0.54	0.51	0.48	0.45
8	0.93	0.88	0.83	0.78	0.74	0.70	0.67	0.64	0.61	0.59	0.54	0.51	0.47	0.44	0.42
9	0.93	0.86	0.81	0.76	0.71	0.68	0.64	0.61	0.58	0.56	0.51	0.47	0.44	0.41	0.38
10	0.92	0.85	0.79	0.74	0.69	0.65	0.61	0.58	0.55	0.53	0.48	0.44	0.41	0.38	0.36
11	0.91	0.83	0.77	0.71	0.67	0.63	0.59	0.56	0.53	0.50	0.45	0.42	0.38	0.36	0.33
12	0.90	0.82	0.75	0.69	0.65	0.60	0.57	0.53	0.50	0.48	0.43	0.39	0.36	0.34	0.31
13	0.89	0.81	0.74	0.68	0.63	0.58	0.54	0.51	0.48	0.45	0.41	0.37	0.34	0.32	0.29
14	0.88	0.79	0.72	0.66	0.61	0.56	0.52	0.49	0.46	0.43	0.39	0.35	0.32	0.30	0.28
15	0.88	0.78	0.70	0.64	0.59	0.54	0.51	0.47	0.44	0.42	0.37	0.34	0.31	0.28	0.26
16	0.87	0.77	0.69	0.63	0.57	0.53	0.49	0.45	0.43	0.40	0.36	0.32	0.29	0.27	0.25
17	0.86	0.76	0.68	0.61	0.56	0.51	0.47	0.44	0.41	0.38	0.34	0.31	0.28	0.26	0.24
18	0.85	0.75	0.66	0.60	0.54	0.49	0.46	0.42	0.40	0.37	0.33	0.30	0.27	0.25	0.23
19	0.85	0.74	0.65	0.58	0.53	0.48	0.44	0.41	0.38	0.36	0.32	0.28	0.26	0.24	0.22
20	0.84	0.72	0.64	0.57	0.51	0.47	0.42	0.40	0.37	0.34	0.30	0.27	0.25	0.23	0.21
21	0.83	0.71	0.63	0.56	0.50	0.45	0.41	0.38	0.36	0.33	0.29	0.26	0.24	0.22	0.20
22	0.83	0.70	0.61	0.54	0.49	0.44	0.40	0.37	0.35	0.32	0.28	0.25	0.23	0.21	0.19
23	0.82	0.69	0.60	0.53	0.48	0.43	0.39	0.36	0.34	0.31	0.27	0.25	0.22	0.20	0.19
24	0.81	0.68	0.59	0.52	0.47	0.42	0.38	0.35	0.33	0.30	0.27	0.24	0.21	0.19	0.18
25	0.80	0.67	0.58	0.51	0.46	0.41	0.37	0.34	0.32	0.29	0.26	0.23	0.20	0.18	0.17

[a] Computed by $100/(100 - P_T + E_T P_T)$, or $100/(100 - P_B + E_B P_B)$, as presented in Chapter Five. Use this formula for larger percentages.
[b] Used to convert equivalent passenger car volumes to actual mixed traffic; use reciprocal of these values to convert mixed traffic to equivalent passenger cars.
[c] From Table 10.4 or Table 10.5.
[d] Trucks and buses should not be combined in entering this table where separate consideration of buses has been established as required, because passenger car equivalents differ.

DIAMOND INTERCHANGES AND
  CROSS CONNECTIONS

Where the interchange is a diamond type, the ramp connection with the local systems of roads may well control the ramp's overall capacity. It normally is analyzed as an intersection at grade, in accordance with procedures outlined in Chapter Six. This will also be the case with cross connections to and from parallel service roads.

Alinement

Restrictive alinement may well exist on ordinary multilane highways, its effect being reflected in lower average highway speeds. As in the freeway case, only approximate measures are available of the influence of this factor on operating speeds and volumes carried; they are incorporated directly into the computational criteria.

Traffic Interruptions and Interferences

By definition, traffic on all highways except freeways is subject to interruption, although the degree varies widely. Fixed traffic interruptions obviously will influence both operating speeds and capacity adversely. These fixed traffic interruptions on the roadway include signalized intersections, stop signs, railroad grade crossings, and the like. Even under free-flow conditions, all of the vehicles will be required to stop at the stop signs, and many will have to stop for the other interruptions; this stopping or slowing creates significantly different operating conditions, if present to an appreciable degree. Alongside the roadway, a variety of other elements, such as strip development, may produce additional interference.

Every fixed interruption accommodates a certain maximum amount of traffic. In particular, every signalized intersection at grade has a capacity based on normal intersection considerations, as discussed in Chapter Six. However, where intersections at grade occur infrequently along a noncontrolled-access multilane road their restrictive effect generally is relatively small (much less than on a typical urban arterial street), because their capacity almost always exceeds the through roadway service volume level being considered in any practical problem. This is true for two main reasons—the

service volume is quite low in relation to through roadway capacity, to provide an acceptable level of service for rural conditions, and the signalized intersection often has a very low percentage of the green time assigned to the cross road, reducing to a minimum the number of vehicles that are detained by it.

Where full access control does not exist, roadside development will occur to a greater or lesser extent. Many highways having partial, rather than complete, access control are classed as expressways. These roads, mostly built in recent years, still allow a small number of highway entrances and intersections at grade at low-volume points, but few permit significant access from roadside businesses and other properties. Such highways fall within the scope of Chapter Nine.

On the other hand, where significant roadside development exists which has direct access to and from the roadway, a different situation exists. It is generally agreed that numerous friction points along a roadway bring about reduction in the traffic-carrying capabilities of the route. Level of service certainly is adversely affected. The findings of one recent study (1) of the delay and congestion caused by commercial roadside development indicate that both average and operating speed on a commercially developed section can be predicted in terms of roadway traffic volume. However, insufficient data have been gathered as yet to accurately predict the loss of capacity, if any, because of the lack of access control. Often, enforced low speed limits are established on such highways for safety reasons. Such limits, if lower than drivers would tend to establish themselves, may permit a larger traffic volume to move, but at a poorer level of service than might occur without the speed control. It may be that capacity, which by definition is obtained under restricted operation at about 30 mph, is little affected if roadside development is relatively light; here the development is mainly a hazard to safety. On the other hand, where development is continuous, operation may be much like that on a city street. The influence of this roadside friction on selection of the traffic determination procedures used depends on its severity, in

terms of its frequency and its effect on speeds.

In general, on multilane highway sections where the number of fixed traffic interruptions are few (greater than 1 mile apart) and other interferences between are largely absent, the presence of these interruptions influences the operating speeds and the maximum service volumes over the section chiefly in terms of restricted flow at or near the interruptions. Operating conditions between fixed interruptions generally retain the characteristics of uninterrupted flow. For the entire highway section, the major consequences are as follows: (1) operating speeds at all volume levels are somewhat lower than for uninterrupted flow because of the necessity for traffic to slow or stop, and (2) more of the traffic travels in platoons, caused by their grouping at the interruptions. These platoons may be of regular or irregular sequence and size, depending on the nature of the interruption. Maximum volumes are limited by the capacity of the most critical point (normally the traffic interruption), although the influence distance of individual interruptions may be relatively limited.

In a relative sense, volume has been found to have the same effect on operating conditions for this somewhat interrupted flow as for uninterrupted flow. Although each highway would have its own specific speed-volume relationship, differing from a typical fully-uninterrupted-flow relationship, the same scale of levels of service is used as for interrupted flow; hence, Table 10.1 continues to apply. Operating speed and service volumes must again be expressed in terms of a value relative to a common maximum. For speeds, the maximum remains the free-flow operating speed. For service volumes, the maximum remains the capacity.

In normal practice, for typical design or operational problems where the fixed interruptions (primarily signalized intersections) average more than 1 mile apart and/or where speed limits are 40 mph or greater between interruptions (and attainable, indicating that the influence of roadside development is not great), it is generally considered reasonable to apply the procedures which follow for uninterrupted flow directly, with-

out detailed consideration of the interruptions. Admittedly, the true capacity at the interruption point of such a roadway section is reduced in proportion to the amount of interruption time in the total time available. For instance, even where only a single signal is present on a long section, the capacity at that point is reduced in proportion to the amount of red time in the total cycle time. But such effects are generally localized on ordinary highways, due to the influence of traffic entering and leaving at other nearby unsignalized points, and do not seriously restrict the section as a whole. At typical design levels of service their influence on speed is relatively small, volumes carried are little affected, and few if any drivers encounter undue delay. At the other extreme a typical rural railroad grade crossing may be a serious impediment to all traffic for several minutes, producing considerable delay to drivers arriving during that period, but over the hour's time on which these criteria are based, again may produce little apparent overall effect.

If, however, there are obviously significant and continuously occurring slowdowns in a particular section, it becomes essential to give special consideration to the capacity of that section and to consider the capabilities of nearby sections as related to those capacities. For example, where the highway in question crosses a more important highway, and has less than half of the total signal time as green time as a result, a significant restriction might exist. Or, where a regularly occurring train movement at a grade crossing conflicts with a peak-period traffic flow which occurs simultaneously, the effect on level of service for a short period may be greatly magnified and require special consideration.

Where fixed interruptions are frequent (more than one per mile), or if speed limits must be restricted to 35 mph or below, indicating substantial roadside interference, the characteristics of the traffic flow are changed too completely for uninterrupted-flow criteria to be applicable even in modified form. The restrictive nature of the operation, produced by frequent signals and other interruptions normally associated with a highway requiring signalization, results in entirely

Traffic interruptions from roadside development and intersections on this divided highway without control of access restrict service volumes.

different, and less consistent, speed-volume relationships. Such highways should be analyzed as urban arterials, as covered later in this chapter. Suburban highways with "strip" development frequently fall into this category.

COMPUTATION PROCEDURES FOR MULTILANE HIGHWAYS WITHOUT ACCESS CONTROL

The generalized procedures first described in Chapter Four, and applied to freeways in Chapter Nine, are equally applicable to more ordinary highways. Again, the procedure must be divided into two parts involving, first, determination of capacities, service volumes, and levels of service of basic individual near-uniform highway subsections, followed by development of overall measures for sections of substantial length formed by several subsections combined.

Basic Uniform Multilane Subsections

Just as was the case for freeways in Chapter Nine, operating speed and the service or demand volume/capacity ratio (v/c ratio) are the basic measures used to identify levels of service on ordinary highways with uninterrupted flow. Table 10.1 summarizes limiting values of these measures defining levels of service; these provide the base for most computations.

Figure 10.1 shows these basic relationships graphically. It is similar to the typical operating speed-volume chart shown earlier, in Figure 3.36, except that the abscissa here represents the service or demand volume/capacity ratio rather than absolute volume. Therefore, it can be used for any highway whose capacity can be computed, rather than representing only highways with ideal conditions. It is useful in problems where interpolation is involved and where a quick visual analysis, or a check of results, is needed. The
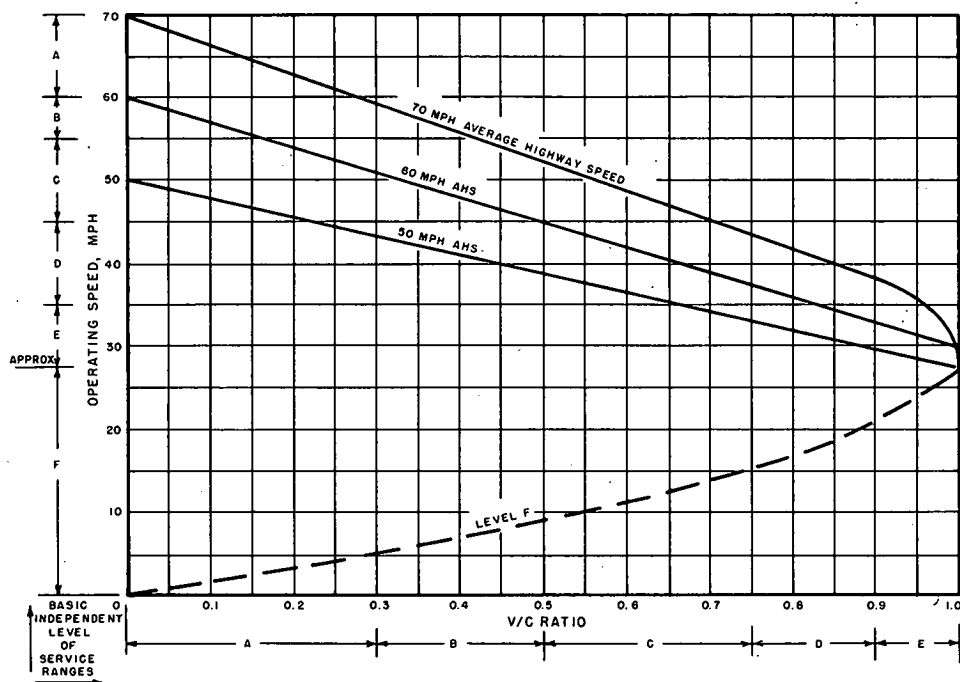
Figure 10.1. Relationships between v/c ratio and operating speed, in one direction of travel, on multilane rural highways, under uninterrupted flow conditions.

basic limiting values of operating speed and v/c ratio which identify the several levels of service are shown on the chart.

Capacity, service volume, and level of service determination procedures for ordinary multilane highways are nearly identical with those for freeways, the differences lying primarily in the basic criteria used, rather than in the methods of use. The procedure is somewhat simplified as compared to that for freeways in that per-lane volumes remain the same no matter how many lanes there may be. However, because lower design standards are likely, greater use of adjustment factors can be expected. Use is made of Table 10.1 (or Fig. 10.1). The procedures are as follows:

CAPACITY (TOTAL FOR ONE DIRECTION) UNDER PREVAILING CONDITIONS

Determine directly, by the standard method for uninterrupted flow. That is,

multiply 2,000 passenger cars per lane per hour by the number of lanes and by the applicable adjustment factors. The truck adjustment used must be that for capacity. Consider intercity buses separately if volumes are large or grades are heavy.

$$c = 2,000 \ N \ W \ T_c$$

in which

$c =$ capacity (mixed vehicles per hour, total for one direction);

$N =$ number of lanes (in one direction);

$W =$ adjustment for lane width and lateral clearance, from Table 10.2 (shoulder adjustment may be necessary, see Chapter Five); and

$T_c =$ truck factor *at capacity*, from Table 10.3b for overall highway section, or Table 10.6 for specific individual grades (intercity bus factor, B_c, may be applied separately, see text).

SERVICE VOLUMES (TOTAL FOR ONE
DIRECTION)

As before, several different procedures are available for use in determining the service volume for a given level of service; selection of the appropriate method depends on the particular data already at hand. In any case, it is important to check the results to confirm that *both* the volume and operating speed criteria for the desired level of service are met, with due consideration for the prevailing average highway speed.

Computed Directly from Capacity Under Ideal Conditions.—Follow the same procedure as that for capacity under prevailing conditions with two exceptions; namely, (1) utilize the adjustments for the appropriate level of service, where different from those for capacity, and (2) apply the appropriate v/c ratio for the level of service desired. (Where alinement is less than ideal, use of the applicable working v/c ratio from Table 10.1, or reference to the appropriate average highway speed curve in Figure 10.1, will help to assure a balance with the operating speed limitation.)

$$SV = 2,000 \, N \, (v/c) \, W \, T_L$$

in which

SV = service volume (mixed vehicles per hour, total for one direction);

N = number of lanes (in one direction);

v/c = volume-capacity ratio, obtained from Table 10.1 (or Fig. 10.1);

W = adjustment for lane width and lateral clearance, from Table 10.2 (shoulder adjustment may be necessary, see Chapter Five); and

T_L = truck factor *at given level of service*, from Table 10.3b for overall highway sections, or Table 10.6 for specific individual grades (intercity bus factor, B_L, may be applied separately, see text).

Confirm attainment of desired level of service by checking resulting operating speed from Figure 10.1, for the given average highway speed, to make sure that the requirement for the given level is met.

Computed from Maximum Service Volume for Ideal Conditions.—(Suitable only where alinement is ideal; that is, for 70-mph average highway speed.) The procedure is identical with the preceding, except that the "maximum service volume" for the number of lanes and level of service desired (from Table 10.1) is used in place of the basic value adjusted by a v/c ratio.

$$SV = MSV \, W \, T_L$$

in which *MSV* is the maximum service volume, in passenger cars per hour, from Table 10.1; and *SV*, *W* and T_L are as before.

Confirm attainment of the desired level of service by checking the resulting operating speed from Figure 10.1, for the given average highway speed.

CAUTION: Use of this method is not appropriate where restricted average highway speeds exist, because it does not make use of the v/c ratio in which the influence of such restrictions is incorporated.

Computed from Capacity Under Prevailing Conditions.—Multiply the capacity obtained under ideal conditions by the v/c ratio obtained from Table 10.1 (or Fig. 10.1) for the level of service desired. (As for ideal conditions, consider use of the working v/c ratio where average highway speed is restricted.) Also, convert the truck adjustment, if used, to that for the level of service involved, rather than capacity.

$$SV = c \, (v/c) \, (T_L/T_c)$$

in which c is the capacity (mixed vehicles per hour, total for one direction), as computed for ideal conditions, and *SV*, v/c, T_L, and T_c are as before.

Confirm attainment of the desired level of service by checking the resulting operating speed from Figure 10.1, for given average highway speed.

Determined from Level of Service Limits.—In the design of a new multilane highway, where a specific level of service has been established in advance, service volumes in passenger cars per hour can be read directly from Table 10.1, provided design conditions are to be ideal. In the more typical case, where alinement or other conditions are less than ideal, Table 10.1 can be used to determine the limiting v/c ratio. From the controlling ratio, the service volume can be determined once capacity is computed (Fig. 10.1 can also be used).

Where a proposed design is already under consideration, the v/c ratio here obtained can be compared with that for the proposed design, to determine its adequacy.

LEVEL OF SERVICE

Determination of the level of service provided by any uncontrolled-access multilane highway, existing or proposed, under uninterrupted flow conditions, while accommodating a given demand volume, requires the same partially "trial-and-error" procedure as described in Chapter Nine for freeways. Here, however, the only "problem" element whose value is dependent on the unknown, level of service, is the truck factor.

The steps are as follows:

(a) Establish a "base volume" for level of service determination, through the same procedure as described under "Service Volumes—Computed Directly from Capacity Under Ideal Conditions," except that no v/c ratio is applied.

$$\text{Base volume} = 2,000\ N\ W\ T_L$$

in which N and W are as before and T_L is the assumed truck factor.

(b) Divide the average demand volume by the "base volume" obtained in (a) to determine the approximate v/c ratio. (Conversion of demand volume to equivalent passenger cars is not necessary, because Step (a) has converted the base to mixed traffic.)

(c) Reinspect Table 10.1 (or Fig. 10.1), if the operating speed was known in advance, to establish the level of service from the controlling factor, either operating speed or basic v/c ratio.

If the operating speed was not known, enter Table 10.1 for the appropriate conditions and determine operating speed. Or, enter Figure 10.1 on the v/c ratio scale, select the appropriate curve for the given average highway speed under consideration, and read the operating speed. Establish the level of service from the controlling factor, either operating speed or basic v/c ratio.

(d) Recompute, using the revised choice of truck factor, if the initial assumption of level of service proves incorrect.

Combined Analysis of Subsections Composing Ordinary Multilane Highway Sections Without Access Control

On ordinary multilane highways, undivided and/or without access control, the likelihood of variations in characteristics over any substantial distance is considerably greater than is the case with modern freeways. Hence, the need to establish weighted average levels of service, combining the influence of several subsections of differing characteristics, may develop more frequently. In overall concept the procedures described in Chapter Nine are appropriate here as well; reference, therefore, should be made to them. However, certain modifications are required; these are described here.

First, regarding units, most problems can be computed equally well in terms of either passenger cars per hour or mixed vehicles per hour. Only where ramp junctions or weaving sections are involved (a more unlikely situation than on freeways) does the mixed traffic approach seem more desirable; most restrictions typically found on ordinary multilane highways, including at-grade intersections, can be handled equally well either way. The important consideration is that consistency be maintained throughout any problem and any comparisons with other analysis results.

The same two typical problems exist: either examination of an existing highway to see if it contains relatively restrictive points, or design of a new highway free of such restrictions. As before, the desirable goal is balanced quality of operation throughout. Because of the many potential restrictions involved throughout, however, this goal is less likely to be fully attained. As before, as long as capacity is not exceeded by demand at any point, overall level of service will be better on a highway with only one or a few service volume restrictions than on one with many.

Typical examples of the procedure used follow in the next section. It should be noted that although "weighted capacity" values remain largely meaningless within any preestablished section, the fact that variation exists which would suggest weighting may make it desirable to reexamine the propriety of the section limits. That is, if difficulty is

Multilane highway without access control, fulfilling function of both traffic and land use service, is subject to the frictions characteristic of "strip" or "ribbon" development.

experienced in describing the overall capacity of a section, it may well be because so many access points exist (including important intersecting highways) between the preestablished terminal points that analysis as one section is, in itself, unfeasible. Again, "weighted service volumes" may have some utility for specialized purposes.

Wherever overall analysis does appear feasible, a "weighted level of service" can be developed. This follows the same general steps as described in Chapter Nine for freeways. Where feasible, weighted average operating speeds and v/c ratios are developed for the section, from those for the component subsections, with weighting proportional to subsection length. These are then compared with limiting values in Table 10.1 (or Fig. 10.1), and a further check is made, using the most critical v/c ratio, to make sure that capacity is not exceeded at any point. Where v/c ratios and operating speeds are not conveniently available, the graphical method described in Chapter Nine will be more suitable.

In the case of signalized intersections and other "point" restrictions, no length is available for use in weighting. In the rural case,

where signals occur infrequently, and usually have adequate capacity to handle typical rural service volumes without difficulty, they usually can be omitted from the weighting procedure and interpreted separately to determine whether they meet the level of the neighboring highway. Where, however, any signal or other point restriction is significant, an approximate length should be assigned based on the apparent influence distance. In the case of point or short-distance lateral clearance restrictions, such as narrow bridges, in the absence of more specific knowledge, an influence distance equivalent to 5 sec travel time plus the actual length of the restriction can be assumed. For instance, a narrow bridge on a highway with a 50-mph operating speed would have an advance influence distance of $50 \times 1.467 \times 5 = 367$ ft.

Where feasible, it is acceptable to develop an overall weighted level of service for a multilane section containing subsections of differing numbers of lanes; each such subsection must have a minimum of four lanes (two in each direction). The sample weighting problems shown in Example 9.8 are equally representative of the ordinary multilane case.

Typical Problem Solutions—Ordinary Multi-lane Highways

EXAMPLE 10.1

Problem:

Given:

Rural 4-lane highway, undivided and without access control.

10-ft lanes.

2-ft shoulders on right; lateral obstructions 2 ft from edge of shoulder.

Overall long section, in rolling terrain.

Alinement provides 60-mph average highway speed.

7% trucks.

Negligible intercity buses.

Major intersections are infrequent and speeds over 45 mph are possible at low volumes.
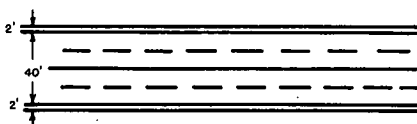
Determine: Service volumes for levels B and E (capacity).

Solution:

(Note: Procedure is similar to that for freeways, the main difference usually being application of more restrictive adjustment factors to correct for conditions less nearly ideal.)

Capacity:

$$c = 2,000 \ N \ W \ T_c$$

where:

$N = 2$.

W, from Table 10.2 for 10-ft lanes with 4-ft clearance, one side $= 0.88$.

T_c, from Table 10.3b for 7% trucks in rolling terrain $= 0.83$.

$c = 2,000 \times 2 \times 0.88 \times 0.83 = 2,920$ vph, total for one direction.

Service Volume B:

$$SV_B = 2,000 \ N \ (v/c) \ W \ T_L$$

where:

$N = 2$.

$v/c = 0.20$, working value from Table 10.1 for level B, AHS $= 60$ mph.

W, as before $= 0.88$.

T_L, as before $= 0.83$.

$SV_B = 2,000 \times 2 \times 0.20 \times 0.88 \times 0.83 = 585$ vph, total for one direction.

Operating speed requirement met by using working v/c value.

EXAMPLE 10.2

Problem:

Given:

Rural 4-lane highway, undivided and without access control.

11-ft lanes.

No shoulders; obstructions located at pavement edge.

Individual grade, 6%, 1 mile long.

Alinement provides 50-mph AHS.

7% trucks.

3% intercity buses.

Demand volume $= 2,100$ vph, in upgrade direction.
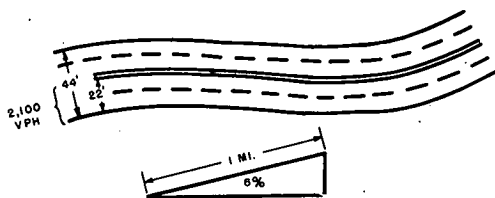
Determine: Level of service being provided on this upgrade.

Solution:

Inspection of Table 10.1, given ordinary 4-lane highway on heavy grade, indicates that operation probably is in level D or poorer.

Assume level D for use in selecting adjustments dependent upon a known level.

On this heavy grade, buses should be considered separately, using bus adjustment factor, B_L.

Base volume $= 2,000 \ N \ W \ T_L \ B_L$

where:

$N = 2$.

W, from Table 10.2 for 11-ft lanes with 0-ft clearance, one side = 0.85.

T_L: From Table 10.4, for level D, given 7% trucks on 6%, 1-mi grade, $E_T = 14$.

From Table 10.6, for $E_T = 14$ and $P_T = 7$, $T_L = 0.52$.

B_L: From Table 10.5, for level D, given 3% buses on 6%, 1-mi grade, $E_B = 4$.

From Table 10.6, for $E_B = 4$ and $P_B = 3$, $B_L = 0.92$.

Base volume = $2,000 \times 2 \times 0.85 \times 0.52 \times 0.92 = 1,627$ vph.

v/c ratio = $2,100/1,627 = 1.29$.

That is, the highway would be overloaded; it could not operate at level D. A computation assuming level E would make no change, because all factors used for level D would be the same at E; overloading would remain.

Hence, level of service is F.

Correction would involve provision of a climbing lane for the heavy truck volume.

TWO-LANE HIGHWAYS

At least two lanes for traffic movement, one in each direction, represent the minimum highway installation normally provided. The decision to provide a 2-lane highway many times is not justified on demand and capacity requirements alone, therefore, but on minimum level of service requirements which justify at least one travel lane in each direction for safety, convenience, and tolerable operating conditions. In terms of mileage, 2-lane highways constitute a majority of the rural main highway system.

Two basic characteristics differentiate traffic operations on 2-lane highways from multilane facilities. First, distribution of traffic by direction has practically no effect on operating conditions, at any given total volume level. Therefore, the capacity and service volumes of 2-lane highways are expressed in total vehicles per hour, regardless of the distribution of traffic by direction. Second, overtaking and passing maneuvers must be made in the traffic lane normally occupied by opposing traffic. Inasmuch as the maintenance of a desired speed requires passing maneuvers, the volume of traffic plus the highway geometrics, which establish available passing sight distance, have a much more significant effect on operating speeds than is the case on multilane roads. Therefore, whenever service volumes are considered for 2-lane roads the corresponding range in available passing sight distance (1,500 ft or greater) must also be considered.

Although the great majority of 2-lane highways do not have control of access, 2-lane highways with access control are by no means rare. Operationally they are not greatly different from uncontrolled highways, except for freedom from the adverse effects of roadside development, because passing and associated potential head-on conflicts remain. In this manual, therefore, no separate criteria are presented; such sections are considered as equivalent to high-type 2-lane roads without access control.

On high-type 2-lane rural highways having uninterrupted flow, the adverse effect of increasing volume on operating speeds is significant even at low volumes and even though ample passing sight distance may be available throughout the roadway section. On 2-lane highways of lower design standards, this effect is less pronounced, but only because the poorer designs do not permit high speeds even at low volumes.

In Chapter Three, typical speed distributions for 2-lane roads are shown in Figure 3.25. Also, typical speed-volume relationships for a range of types of 2-lane roads, with differing average highway speeds and typical related passing sight distance conditions, but otherwise ideal conditions, are shown in Figures 3.37 and 3.40. Those curves, like the criteria in this section, are based on curves developed in the mid-1950's (2), but have been modified to reflect the current higher speeds of even the

Winding 2-lane rural road with steep grades and carrying significant truck volume has greatly restricted capacity.

slowest vehicles, made evident by speed trends studies of recent years (3). It should be noted that, unlike the straight-line speed-volume relationships for freeways and multi-lane highways, 2-lane operating speed-volume relationships take a somewhat wavy form.

LEVELS OF SERVICE

Two-lane highway geometrics primarily affect operating speeds during the free flow representative of level of service A, their effects becoming less significant by the time the maximum volume in this level is reached. Average speeds are most influenced by speed limits in this level also. Within level A, operating speeds must be 60 mph or higher. If passing sight distance is always available volumes may reach 30 percent of capacity. Under ideal conditions a service volume of 400 passenger cars per hour, total for both directions, may be achieved. Approximately 75 percent of the desired passing maneuvers

can be made with little or no delay, the main deterrent, of course, being vehicles traveling in the opposite direction.

Level of service B marks the beginning of stable flow. At the maximum volume limit of this level most of the drivers must govern their speeds according to volume conditions. In terms of passing maneuvers, the average driver may wish to increase the number of passings, but cannot do so due to increased traffic densities. Therefore, most drivers are affected by other vehicles in the traffic stream, although this effect is not yet unreasonable. Operating speeds are 50 mph or above, and volumes may reach 45 percent of capacity with continuous passing sight distance. Volumes of 900 passenger cars per hour, total for both directions, are carried under ideal conditions.

Further increases in volume have a direct effect on operating speeds, independent of highway alinement features. In the limit of level of service C, still stable flow, operating speeds for uninterrupted flow on all 2-lane highways are 40 mph or above, with total volume for both directions reaching 70 percent of capacity with continuous passing sight distance, or 1,400 passenger cars per hour, under ideal conditions.

Unstable flow is approached as operating speeds fall to 35 mph. Volumes carried, total for both directions, may reach 85 percent of capacity with continuous passing sight distance, or 1,700 passenger cars per hour, under ideal conditions. This represents the limiting conditions for level of service D, or the highest volume that can be maintained for short periods of time without a high probability of breakdown in flow.

At level of service E, or capacity, actual operating speeds will usually be in the neighborhood of 30 mph, but may vary considerably. Volumes, total for both directions, under ideal conditions, will be 2,000 passenger cars per hour. Again, as with other highway types, level F represents forced, congested flow with relatively unpredictable characteristics. Operating speeds are less than 30 mph, and volumes are under 2,000 passenger cars total for both directions. Frequently, level E is never attained as volume builds up; instead, a transition into level F occurs directly from level D.

The effect of restricted passing sight distance can be considered in two ways. A highway' with restricted passing provides lower operating speeds at the same volume than one with unrestricted passing. Conversely, maintaining a comparable operating speed when passing is restricted requires lower service volumes. The latter concept is used for 2-lane highways, inasmuch as level of service is expressed consistently throughout this manual mainly in terms of operating speed as the governing control, with the volume limitation as a supplementary control. At any given level of service limit, operating speeds are equal. The effect of passing sight distance restrictions is to lower the service volume at that given level of service. Restricted alinement (lower than ideal average highway speed) also produces this same effect, while also restricting or totally eliminating the ability to attain the higher levels of service.

Table 10.7 gives the scale of operating characteristics established for the various levels of service on 2-lane highways and summarizes general level of service criteria during uninterrupted flow conditions. Included, in addition to operating speeds and basic volume/capacity ratios for ideal alinement, are approximate measures of the influence of passing sight distance, expressed as a percentage of the total section that is adequate (greater than 1,500 ft), and of average highway speeds, on working v/c ratios.

CRITICAL ELEMENTS REQUIRING CONSIDERATION

Lane Width and Lateral Clearance

In the case of 2-lane highways, current design standards vary considerably. Certain modern 2-lane roads, intended for only low volumes, may have lane widths of only 10 ft, rather than the 12-ft ideal value. Some older highways may have only 9-ft lanes, together with restricted lateral clearances. Table 10.8 gives adjustment factors reflecting the combined adverse influence of restricted lane width and lateral clearance on 2-lane highways.

Trucks, Buses, and Grades

The effect of trucks, buses, and grades on 2-lane highways has been discussed in detail in Chapter Five. As on multilane highways, trucks and buses have an influence which must be considered, even in the case of level terrain. However, the influence becomes much greater on grades. It is pointed out in Chapter Five that most typical grades influence operations only when trucks (and sometimes intercity buses) are present, and also states that the effect varies with the length and steepness of the grade as well as the level of service under consideration. It further indicates that the average effect of trucks and buses over a highway section of substantial length differs from that on most individual grades.

Table 10.9a gives average generalized passenger car equivalents of trucks over extended lengths of 2-lane highways, for various terrain conditions and levels of service. Although bus volumes seldom warrant separate consideration, separate equivalents for buses are also given for use where such volumes are significant.

Table 10.9b gives general overall adjustment factors for conversion of mixed demand volumes of trucks and passenger cars over extended lengths of 2-lane highway into equivalent passenger vehicles per hour, based on these overall passenger car equivalents. These can be used in overall analyses of the capabilities of substantial lengths of 2-lane highways, which include downgrades and level portions, as well as upgrades. Where separate consideration of buses appears necessary, Table 10.9b is not appropriate. Rather, the equivalents for buses given in Table 10.9a should be used in conjunction with Table 10.12 to obtain separate adjustment factors.

Where the need is to determine the influence of trucks and buses on specific individual upgrades, the process is more selective. From criteria described in Chapter Five, Table 10.10 has been prepared to present detailed passenger car equivalency factors for trucks at capacity and the several levels of service, on two-lane highways where no climbing lane is provided. Table 10.11 similarly presents passenger car equivalency factors for intercity buses, for use in those

TABLE 10.7—LEVELS OF SERVICE AND MAXIMUM SERVICE VOLUMES ON TWO-LANE HIGHWAYS UNDER UNINTERRUPTED
FLOW CONDITIONS
(NORMALLY REPRESENTATIVE OF RURAL OPERATION)

LEVEL OF SERVICE	TRAFFIC FLOW CONDITIONS			SERVICE VOLUME/CAPACITY (v/c) RATIO							MAXIMUM SERVICE VOLUME UNDER IDEAL CONDITIONS, INCLUDING 70-MPH AHS (PASSENGER CARS, TOTAL, BOTH DIRECTIONS, PER HOUR)
	DESCRIPTION	OPERATING SPEED[a] (MPH)	PASSING SIGHT DISTANCE >1,500 FT (%)	BASIC LIMITING VALUE[a] FOR AHS OF 70 MPH	WORKING VALUE FOR RESTRICTED AVERAGE HIGHWAY SPEED[b] OF						
					60 MPH	50 MPH	45 MPH	40 MPH	35 MPH	
A	Free flow	≥60	100	≲ 0.20	—	—	—	—	—	400
			80	0.18	—	—	—	—	—	
			60	0.15	—	—	—	—	—	
			40	0.12	—	—	—	—	—	
			20	0.08	—	—	—	—	—	
			0	0.04	—	—	—	—	—	
B	Stable flow (upper speed range)	≥50	100	≲ 0.45	≲ 0.40	—	—	—	—	900
			80	0.42	0.35	—	—	—	—	
			60	0.38	0.30	—	—	—	—	
			40	0.34	0.24	—	—	—	—	
			20	0.30	0.18	—	—	—	—	
			0	0.24	0.12	—	—	—	—	
C	Stable flow	≥40	100	≲ 0.70	≲ 0.66	≲ 0.56	≲ 0.51	—	—	1400
			80	0.68	0.61	0.53	0.46	—	—	
			60	0.65	0.56	0.47	0.41	—	—	
			40	0.62	0.51	0.38	0.32	—	—	
			20	0.59	0.45	0.28	0.22	—	—	
			0	0.54	0.38	0.18	0.12	—	—	

D	Approaching unstable flow	≥35	100	≲ 0.85	≲ 0.83	≲ 0.75	≲ 0.67	≲ 0.58	—	1700
			80	0.84	0.81	0.72	0.62	0.55	—	
			60	0.83	0.79	0.69	0.57	0.51	—	
			40	0.82	0.76	0.66	0.52	0.45	—	
			20	0.81	0.71	0.61	0.44	0.35	—	
			0	0.80	0.66	0.51	0.30	0.19	—	
E[c]	Unstable flow	30[d]	Not applicable[e]	≲1.00						2000
F	Forced flow	<30[d]	Not applicable[e]	Not Meaningful[f]						Widely variable (0 to capacity)

[a] Operating speed and basic v/c ratio are independent measures of level of service; both limits must be satisfied in any determination of level.
[b] Where no entry appears, operating speed required for this level is unattainable even at low volumes.
[c] Capacity.
[d] Approximately.
[e] No passing.
[f] Demand volume/capacity ratio may well exceed 1.00, indicating overloading.

TABLE 10.8—COMBINED EFFECT OF LANE WIDTH AND RESTRICTED LATERAL CLEARANCE ON CAPACITY AND SERVICE VOLUMES OF TWO-LANE HIGHWAYS WITH UNINTERRUPTED FLOW

DISTANCE FROM TRAFFIC LANE EDGE TO OBSTRUCTION (FT)	ADJUSTMENT FACTORS W_L AND W_c FOR LATERAL CLEARANCE AND LANE WIDTH[a]															
	OBSTRUCTION ON ONE SIDE ONLY[b]								OBSTRUCTIONS ON BOTH SIDES[b]							
	12-FT LANES		11-FT LANES		10-FT LANES		9-FT LANES		12-FT LANES		11-FT LANES		10-FT LANES		9-FT LANES	
	LEVEL B	LEVEL E[c]	LEVEL B	LEVEL E[c]	LEVEL B	LEVEL E[c]	LEVEL B	LEVEL E[c]	LEVEL B	LEVEL E[c]	LEVEL B	LEVEL E[c]	LEVEL B	LEVEL E[c]	LEVEL B	LEVEL E[c]
6	1.00	1.00	0.86	0.88	0.77	0.81	0.70	0.76	1.00	1.00	0.86	0.88	0.77	0.81	0.70	0.76
4	0.96	0.97	0.83	0.85	0.74	0.79	0.68	0.74	0.92	0.94	0.79	0.83	0.71	0.76	0.65	0.71
2	0.91	0.93	0.78	0.81	0.70	0.75	0.64	0.70	0.81	0.85	0.70	0.75	0.63	0.69	0.57	0.65
0	0.85	0.88	0.73	0.77	0.66	0.71	0.60	0.66	0.70	0.76	0.60	0.67	0.54	0.62	0.49	0.58

[a] Adjustment W_c given for level E, capacity, and W_L for level B; interpolate for others.
[b] Includes allowance for opposing traffic.
[c] Capacity.

TABLE 10.9a—AVERAGE GENERALIZED PASSENGER CAR EQUIVALENTS OF
TRUCKS AND BUSES ON TWO-LANE HIGHWAYS, OVER EXTENDED
SECTION LENGTHS
(INCLUDING UPGRADES, DOWNGRADES, AND LEVEL SUBSECTIONS)

EQUIVALENT	LEVEL OF SERVICE	EQUIVALENT, FOR:		
		LEVEL TERRAIN	ROLLING TERRAIN	MOUNTAINOUS TERRAIN
E_T, for trucks	A	3	4	7
	B and C	2.5	5	10
	D and E	2	5	12
E_B, for buses[a]	All levels	2	4	6

[a] Separate consideration not warranted in most problems; use only where bus volumes are significant.

TABLE 10.9b—AVERAGE GENERALIZED ADJUSTMENT FACTORS FOR TRUCKS[b]
ON TWO-LANE HIGHWAYS, OVER EXTENDED SECTION LENGTHS

PERCENTAGE OF TRUCKS, P_T	TRUCK ADJUSTMENT FACTOR, T								
	LEVEL TERRAIN			ROLLING TERRAIN			MOUNTAINOUS TERRAIN		
	LEVEL OF SERVICE A	LEVELS OF SERVICE B AND C	LEVELS OF SERVICE D AND E[c]	LEVEL OF SERVICE A	LEVELS OF SERVICE B AND C	LEVELS OF SERVICE D AND E[c]	LEVEL OF SERVICE A	LEVELS OF SERVICE B AND C	LEVELS OF SERVICE D AND E[c]
1	0.98	0.99	0.99	0.97	0.96	0.96	0.94	0.92	0.90
2	0.96	0.97	0.98	0.94	0.93	0.93	0.89	0.85	0.82
3	0.94	0.96	0.97	0.92	0.89	0.89	0.85	0.79	0.75
4	0.93	0.95	0.96	0.89	0.86	0.86	0.81	0.74	0.69
5	0.91	0.93	0.95	0.87	0.83	0.83	0.77	0.69	0.65
6	0.89	0.92	0.94	0.85	0.81	0.81	0.74	0.65	0.60
7	0.88	0.91	0.93	0.83	0.78	0.78	0.70	0.61	0.57
8	0.86	0.90	0.93	0.81	0.76	0.76	0.68	0.58	0.53
9	0.85	0.89	0.92	0.79	0.74	0.74	0.65	0.55	0.50
10	0.83	0.87	0.91	0.77	0.71	0.71	0.63	0.53	0.48
12	0.81	0.85	0.89	0.74	0.68	0.68	0.58	0.48	0.43
14	0.78	0.83	0.88	0.70	0.64	0.64	0.54	0.44	0.39
16	0.76	0.81	0.86	0.68	0.61	0.61	0.51	0.41	0.36
18	0.74	0.80	0.85	0.65	0.58	0.58	0.48	0.38	0.34
20	0.71	0.77	0.83	0.63	0.56	0.56	0.45	0.36	0.31

[b] Not applicable to buses where they are given separate specific consideration; use instead Table 10.9a in conjunction with Table 10.12.
[c] Capacity.

TABLE 10.10—PASSENGER CAR EQUIVALENTS OF TRUCKS ON TWO-LANE HIGHWAYS, ON SPECIFIC INDIVIDUAL SUBSECTIONS OR GRADES

GRADE (%)	LENGTH OF GRADE (MI)	PASSENGER CAR EQUIVALENT, E_T (FOR ALL PERCENTAGES OF TRUCKS)		
		LEVELS OF SERVICE A AND B	LEVEL OF SERVICE C	LEVELS OF SERVICE D AND E (CAPACITY)
0–2	All	2	2	2
3	¼	5	3	2
	½	10	10	7
	¾	14	16	14
	1	17	21	20
	1½	19	25	26
	2	21	27	29
	3	22	29	31
	4	23	31	32
4	¼	7	6	3
	½	16	20	20
	¾	22	30	32
	1	26	35	39
	1½	28	39	44
	2	30	42	47
	3	31	44	50
	4	32	46	52
5	¼	10	10	7
	½	24	33	37
	¾	29	42	47
	1	33	47	54
	1½	35	51	59
	2	37	54	63
	3	39	56	66
	4	40	57	68
6	¼	14	17	16
	½	33	47	54
	¾	39	56	65
	1	41	59	70
	1½	44	62	75
	2	46	65	80
	3	48	68	84
	4	50	71	87
7	¼	24	32	35
	½	44	63	75
	¾	50	71	84
	1	53	74	90
	1½	56	79	95
	2	58	82	100
	3	60	85	104
	4	62	87	108

TABLE 10.11—PASSENGER CAR EQUIVALENTS OF INTERCITY BUSES ON TWO-LANE HIGHWAYS, ON SPECIFIC INDIVIDUAL SUBSECTIONS OR GRADES

GRADE[a] (%)	PASSENGER CAR EQUIVALENT[b], E_B		
	LEVELS OF SERVICE A AND B	LEVEL OF SERVICE C	LEVELS OF SERVICE D AND E (CAPACITY)
0–4	2	2	2
5[c]	4	3	2
6[c]	7	6	4
7[c]	12	12	10

[a] All lengths.
[b] For all percentages of buses.
[c] Use generally restricted to grades over ½ mile long.

few cases where bus volumes are heavy and/or grades are heavy.

In practice, the values from these tables normally are not used directly in computations, but are used to enter Table 10.12, which provides truck factors that consider both the passenger car equivalent and the percentage of trucks in the traffic stream. The procedures are described later in this section.

As described in Chapter Five, these truck procedures and adjustments assume "average trucks." Where such an assumption is not valid, special analyses making use of data in Chapter Five may be necessary to obtain a passenger car equivalency factor from Figure 5.6 for use in entering Table 10.12.

Downgrades are of special significance in the case of 2-lane highways, because they are so closely interrelated with upgrades. An upgrade for one direction of flow is, of course, a downgrade for the other direction. Because capacities and service volumes can be quoted only as total volumes for both directions, the effects of the opposing downgrade are necessarily included in any consideration of a particular upgrade. However, the specific nature of these effects has not yet been established.

The following generalizations may be made:

1. On flat grades, the effect of trucks in upgrade and downgrade flows can be taken as the same without appreciable error, although the adverse effect on downgrades is probably actually less.

2. On individual heavy downgrades, where trucks must descend in a low gear for safety, there is increasing feeling that trucks produce nearly as great an adverse effect as on an equivalent upgrade.

3. Where the demand volume is reported separately for each direction of flow, but only an overall percentage of trucks is given, it can be assumed that the percentage applies to both directions individually as well.

Therefore, it can be concluded that only on intermediate grades of about 2 to 4 percent, or at locations where the percentage of trucks in the two directions differs widely, is a significant error possibly introduced into the computations if a single truck factor is applied.

Where local observations indicate that such an error would be introduced, and available data (particularly average truck speeds downgrade) permit refinement, sepa-

TABLE 10.12—ADJUSTMENT FACTORS[a] FOR TRUCKS AND BUSES ON INDIVIDUAL ROADWAY SUBSECTIONS OR GRADES ON TWO-LANE HIGHWAYS (INCORPORATING PASSENGER CAR EQUIVALENT AND PERCENTAGE OF TRUCKS OR BUSES)[b]

PASSENGER CAR EQUIVALENT, E_T OR E_B[c]	TRUCK ADJUSTMENT FACTOR T_c OR T_L (B_c OR B_L FOR BUSES)[d] PERCENTAGE OF TRUCKS, P_T (OR OF BUSES, P_B) OF:														
	1	2	3	4	5	6	7	8	9	10	12	14	16	18	20
2	0.99	0.98	0.97	0.96	0.95	0.94	0.93	0.93	0.92	0.91	0.89	0.88	0.86	0.85	0.83
3	0.98	0.96	0.94	0.93	0.91	0.89	0.88	0.86	0.85	0.83	0.81	0.78	0.76	0.74	0.71
4	0.97	0.94	0.92	0.89	0.87	0.85	0.83	0.81	0.79	0.77	0.74	0.70	0.68	0.65	0.63
5	0.96	0.93	0.89	0.86	0.83	0.81	0.78	0.76	0.74	0.71	0.68	0.64	0.61	0.58	0.56
6	0.95	0.91	0.87	0.83	0.80	0.77	0.74	0.71	0.69	0.67	0.63	0.59	0.56	0.53	0.50
7	0.94	0.89	0.85	0.81	0.77	0.74	0.70	0.68	0.65	0.63	0.58	0.54	0.51	0.48	0.45
8	0.93	0.88	0.83	0.78	0.74	0.70	0.67	0.64	0.61	0.59	0.54	0.51	0.47	0.44	0.42
9	0.93	0.86	0.81	0.76	0.71	0.68	0.64	0.61	0.58	0.56	0.51	0.47	0.44	0.41	0.38
10	0.92	0.85	0.79	0.74	0.69	0.65	0.61	0.58	0.55	0.53	0.48	0.44	0.41	0.38	0.36
11	0.91	0.83	0.77	0.71	0.67	0.63	0.59	0.56	0.53	0.50	0.45	0.42	0.38	0.36	0.33
12	0.90	0.82	0.75	0.69	0.65	0.60	0.57	0.53	0.50	0.48	0.43	0.39	0.36	0.34	0.31
13	0.89	0.81	0.74	0.68	0.63	0.58	0.54	0.51	0.48	0.45	0.41	0.37	0.34	0.32	0.29
14	0.88	0.79	0.72	0.66	0.61	0.56	0.52	0.49	0.46	0.43	0.39	0.35	0.32	0.30	0.28
15	0.88	0.78	0.70	0.64	0.59	0.54	0.51	0.47	0.44	0.42	0.37	0.34	0.31	0.28	0.26
16	0.87	0.77	0.69	0.63	0.57	0.53	0.49	0.45	0.43	0.40	0.36	0.32	0.29	0.27	0.25
17	0.86	0.76	0.68	0.61	0.56	0.51	0.47	0.44	0.41	0.38	0.34	0.31	0.28	0.26	0.24
18	0.85	0.75	0.66	0.60	0.54	0.49	0.46	0.42	0.40	0.37	0.33	0.30	0.27	0.25	0.23
19	0.85	0.74	0.65	0.58	0.53	0.48	0.44	0.41	0.38	0.36	0.32	0.28	0.26	0.24	0.22
20	0.84	0.72	0.64	0.57	0.51	0.47	0.42	0.40	0.37	0.34	0.30	0.27	0.25	0.23	0.21
22	0.83	0.70	0.61	0.54	0.49	0.44	0.40	0.37	0.35	0.32	0.28	0.25	0.23	0.21	0.19
24	0.81	0.68	0.59	0.52	0.47	0.42	0.38	0.35	0.33	0.30	0.27	0.24	0.21	0.19	0.18
26	0.80	0.67	0.57	0.50	0.44	0.40	0.36	0.33	0.31	0.29	0.25	0.22	0.20	0.18	0.17
28	0.79	0.65	0.55	0.48	0.43	0.38	0.35	0.32	0.29	0.27	0.24	0.21	0.19	0.17	0.16
30	0.78	0.63	0.53	0.46	0.41	0.36	0.33	0.30	0.28	0.26	0.22	0.20	0.18	0.16	0.15
35	0.75	0.60	0.49	0.42	0.37	0.33	0.30	0.27	0.25	0.23	0.20	0.17	0.16	0.14	0.13
40	0.72	0.56	0.46	0.39	0.34	0.30	0.27	0.24	0.22	0.20	0.18	0.15	0.14	0.12	0.11
45	0.69	0.53	0.43	0.36	0.31	0.27	0.25	0.22	0.20	0.19	0.16	0.14	0.12	0.11	0.10
50	0.67	0.51	0.40	0.34	0.29	0.25	0.23	0.20	0.18	0.17	0.15	0.13	0.11	0.10	0.09
55	0.65	0.48	0.38	0.32	0.27	0.24	0.21	0.19	0.17	0.16	0.13	0.12	0.10	0.09	0.08
60	0.63	0.46	0.36	0.30	0.25	0.22	0.19	0.17	0.16	0.15	0.12	0.11	0.10	0.09	0.08
65	0.61	0.44	0.34	0.28	0.24	0.21	0.18	0.16	0.15	0.14	0.12	0.10	0.09	0.08	0.07
70	0.59	0.42	0.33	0.27	0.22	0.19	0.17	0.15	0.14	0.13	0.11	0.09	0.08	0.07	0.07
75	0.57	0.40	0.31	0.25	0.21	0.18	0.16	0.14	0.13	0.12	0.10	0.09	0.08	0.07	0.06
80	0.56	0.39	0.30	0.24	0.20	0.17	0.15	0.14	0.12	0.11	0.10	0.08	0.07	0.07	0.06
90	0.53	0.36	0.27	0.22	0.18	0.16	0.14	0.12	0.11	0.10	0.09	0.07	0.07	0.06	0.05
100	0.50	0.34	0.25	0.20	0.17	0.14	0.13	0.11	0.10	0.09	0.08	0.07	0.06	0.06	0.05

[a] Computed by $100/(100 - P_T + E_T P_T)$, or $100/(100 - P_B + E_B P_B)$, as presented in Chapter Five. Use this formula for larger percentages.

[b] Used to convert equivalent passenger car volumes to actual mixed traffic; use reciprocal of these values to convert mixed traffic to equivalent passenger cars.

[c] From Table 10.10 or Table 10.11.

[d] Trucks and buses should not be combined in entering this table where separate consideration of buses has been established as required, because passenger car equivalents differ.

rate truck factors can be introduced into the computation for the two directions of flow. When this is done it is important that both truck volume percentages used be of the total flow, not of the separate directional flows. Reference to the procedures in Chapter Five will here be necessary to develop a passenger car. equivalent for the downgrade flow, based on average downgrade speeds. Example 10.5 includes a demonstration of this procedure.

Ramp Entrances and Exits

Many, diamond interchange ramps and cross connections have entrance or exit terminals on a 2-lane, noncontrolled-access roadway, the junction forming either a 90° angle or at least an appreciable angle. Such ramp terminals, often signalized, will perform the same as a normal street intersection. Therefore, such ramp entrances and exits should be analyzed as simple street intersections in accordance with the procedures established in Chapter Six.

Occasional cloverleaf or direct connection ramps, however, may connect to 2-lane highways at "flat" angles. Such junctions should be analyzed by the methods in Chapter Eight, with consideration given to the fact that, because by-lane capacities cannot be quoted for 2-lane roads, assumptions regarding traffic distribution between directions will be necessary.

Alinement

On 2-lane highways the adverse effects of alinement may be substantial, because many such existing highways, and even some new designs, involve rather low standards. As before, this effect is reflected in the average highway speed. For 2-lane highways, this effect has been studied in some detail. The influence on operating speeds and volumes carried is incorporated directly into the computational criteria.

Traffic Interruptions and Interferences

Just as is the case with multilane highways, fixed traffic interruptions have an adverse effect on 2-lane highway operation, and must be considered if present to an appreciable extent. As before, an occasional signalized intersection at grade will not

materially affect the better levels of service on rural 2-lane roads, because relatively few cars are stopped and the intersection capacity, as computed by the methods of Chapter Six, greatly exceeds the service volumes associated with these better levels. Where volumes are greater, or intersections are located close together, the effect may become significant.

Noncontrol of access and roadside development are significant interferences on 2-lane roads. Although some controlled-access 2-lane highways have been constructed, roadside frictions are present along almost all 2-lane roads. Their effects, although similar to those previously described for multilane highways, are likely to be more serious on 2-lane facilities because the roadside lanes, typically most influenced by turbulence, are the *only* lanes.

Where interruptions and interferences are present, but are not restrictive enough to result in signalized intersections closer than 1 mile apart, or in speed limits or attainable speeds between interruptions below 35 mph, the procedures described here for uninterrupted flow are considered appropriate in the typical case. Where these limits are not met, the highway normally should be analyzed as an urban arterial street, as described later. Again, as with multilane highways, there are exceptions to this general rule where judgment must be used to determine the significance to the average driver of the interruptions present in the particular case.

One important point should be noted, however. On 2-lane highways interruptions due to momentary stops, stalling, vehicular breakdowns, accidents, and the like, may well have a much greater effect on operations than would similar incidents on multilane facilities, because the chance of complete blockage of one or both directions of flow is much greater. The average overall effect of the more common of such impediments, most of which are of brief duration and occur daily, is reflected in the level of service data that have been presented. The consequences of a complete blockage should be considered seriously, however, in weighing the advantages of a 2-lane versus a 4-lane design under borderline conditions.

COMPUTATION PROCEDURES FOR TWO-LANE HIGHWAYS

The generalized procedures described in Chapter Four are fully as applicable to 2-lane as to multilane highways. Because of the several inherent variables involved in 2-lane highway operation, as well as the effect of external influences, the several subsections composing a typical 2-lane highway are likely to have significant variation in prevailing conditions. Again, then, procedures are required for determining both the basic capabilities of individual sections and the overall capabilities of their combinations into sections of appreciable length.

Basic Uniform Two-Lane Subsections

As with higher types of highways previously described, the service or demand volume/capacity ratio remains the basic volume measure which is related to operating speed and equivalent level of service, on two-lane highways. Because of the influence of percentage of available passing sight distance, and of the wide range of possible average highway speeds, the fundamental Table 10.7 is more complex than the similar tables for higher types of highways. Here, therefore, graphical charts are often useful; however, no single chart can serve as the all-inclusive fundamental base for computations. Instead, a series of charts is necessary, relating the service or demand volume/capacity ratio to operating speed for a variety of highway types, as presented in Figures 10.2a through 10.2f.

Each of the charts represents a specific average highway speed, and includes coverage of the full range of percentages of available passing sight distance. Thus, average highway speed is the control used in selection of the proper chart for a specific problem.

The basic values of both the v/c ratio and the operating speed which establish the limits of the several levels of service are shown in Figure 10.2a, which includes the curve of ideal conditions. On the remaining charts only the operating speed values are shown, inasmuch as they control. It should be noted that the value shown for the approximate limit of level E varies from chart to chart; it may fall as low as about 25 mph under the poorest alinement conditions.

Procedures for determination of capacity and service volumes for 2-lane highways, for establishment of level of service provided, and for related determinations, are similar in concept to those for multilane highways. However, those involving levels of service are necessarily somewhat more complex, due to the influence of available passing sight distance and to the increased likelihood of restricted alinement. Also, there are more cases of variation between adjustment factors for capacity and for levels of service. The procedural steps are as follows.

CAPACITY (TOTAL IN BOTH DIRECTIONS) UNDER PREVAILING CONDITIONS

Determined directly, by the standard method for all highway types. Here specifically, multiply 2,000 passenger cars total for both directions by the several appropriate adjustment factors, including those for lane width and lateral clearance, shoulders, and trucks on grades. Consider intercity buses separately where their volumes are large or grades are heavy. Where adjustment factors differ for capacity and for service volumes, make sure that the capacity adjustment is utilized. (No adjustment for available passing sight distance is required because passing is not feasible at capacity.)

$$c = 2,000 \ W_c \ T_c$$

in which

$c =$ capacity (mixed vehicles per hour, total in both directions);

$W_c =$ adjustment for lane width and lateral clearance at capacity, from Table 10.8 (shoulder adjustment may be necessary, see Chapter Five); and

$T_c =$ truck factor at capacity, from Table 10.9b for overall highway sections, or Table 10.12 for specific individual grades (intercity bus factor, B_L, may be applied, see text).

SERVICE VOLUMES (TOTAL IN BOTH DIRECTIONS)

Again, as with higher types of highways, several different procedures are available for use in determining the service volume for a given level of service; selection of the ap-
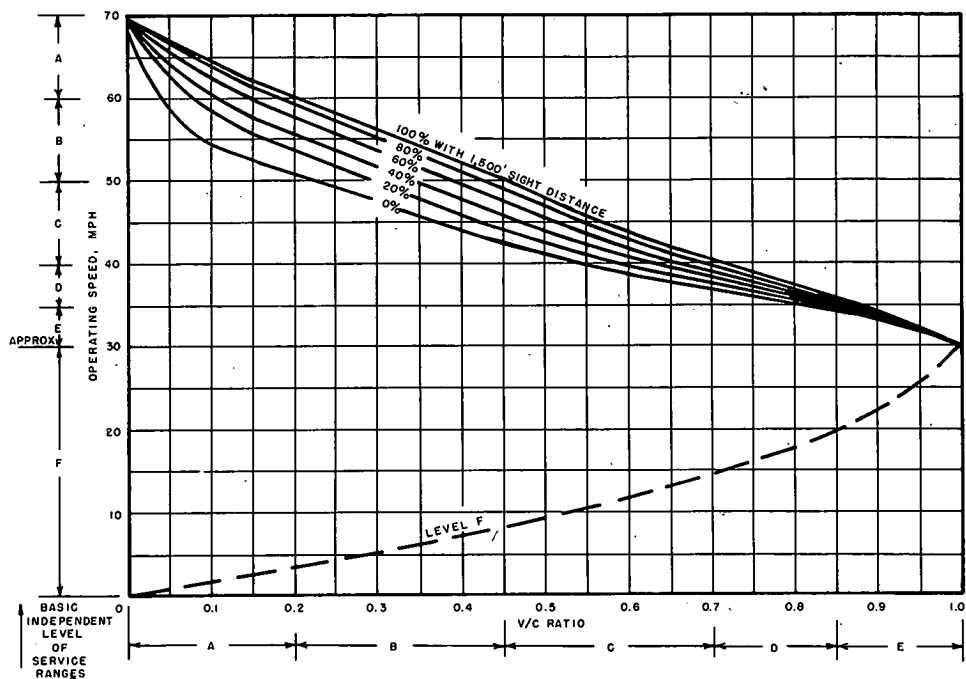
Figure 10.2a. Relationships between v/c ratio and operating speed, overall for both directions of travel, on two-lane rural highways with average highway speed of 70 mph, under uninterrupted flow conditions.
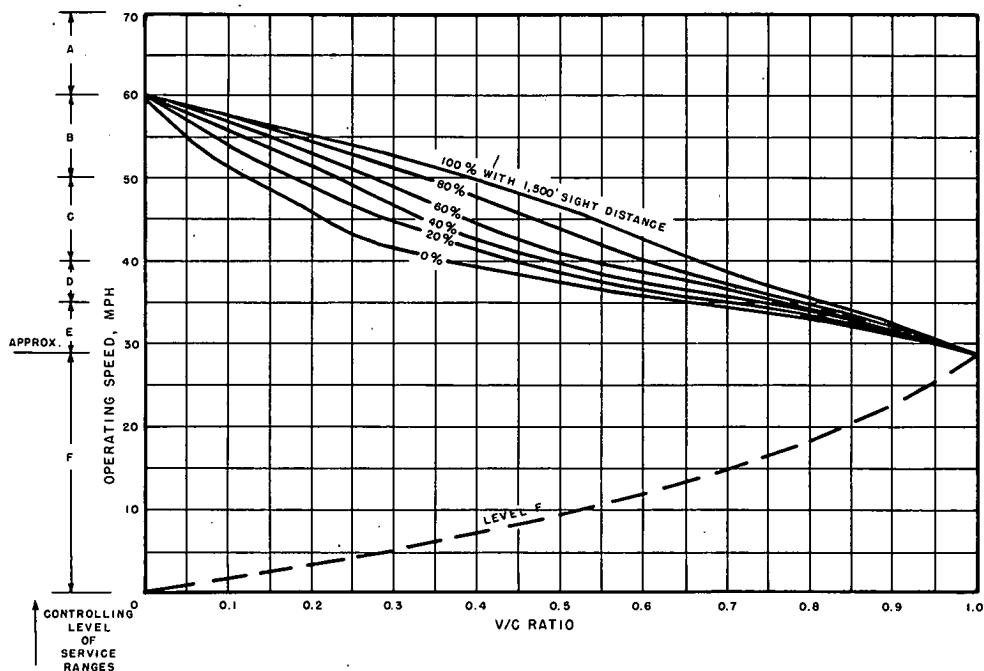
Figure 10.2b. Relationships between v/c ratio and operating speed, overall for both directions of travel, on two-lane rural highways with average highway speed of 60 mph, under uninterrupted flow conditions.
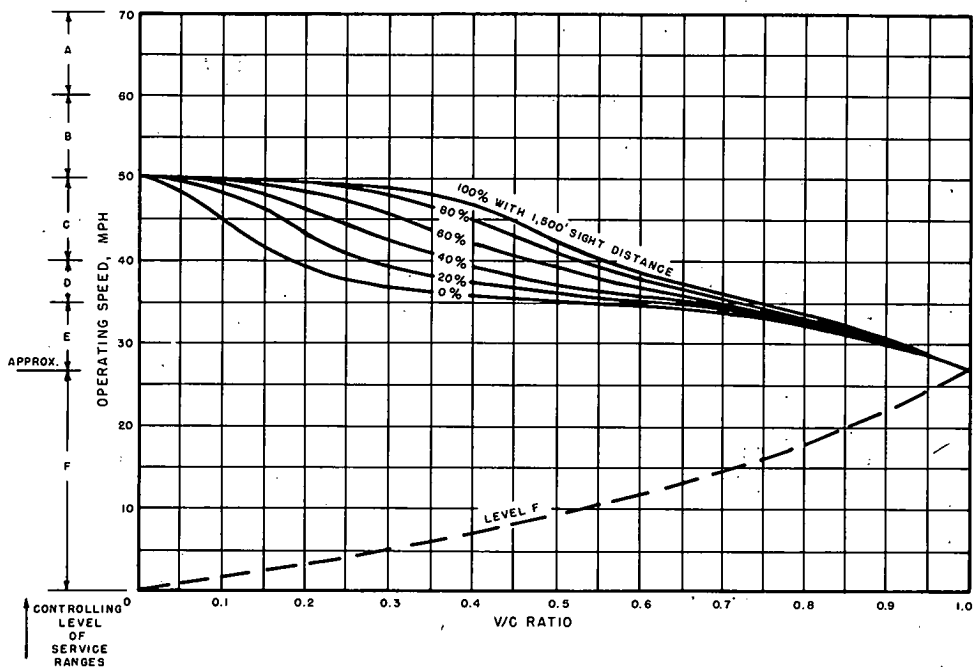
Figure 10.2c. Relationships between v/c ratio and operating speed, overall for both directions of travel, on two-lane rural highways with average highway speed of 50 mph, under uninterrupted flow conditions.
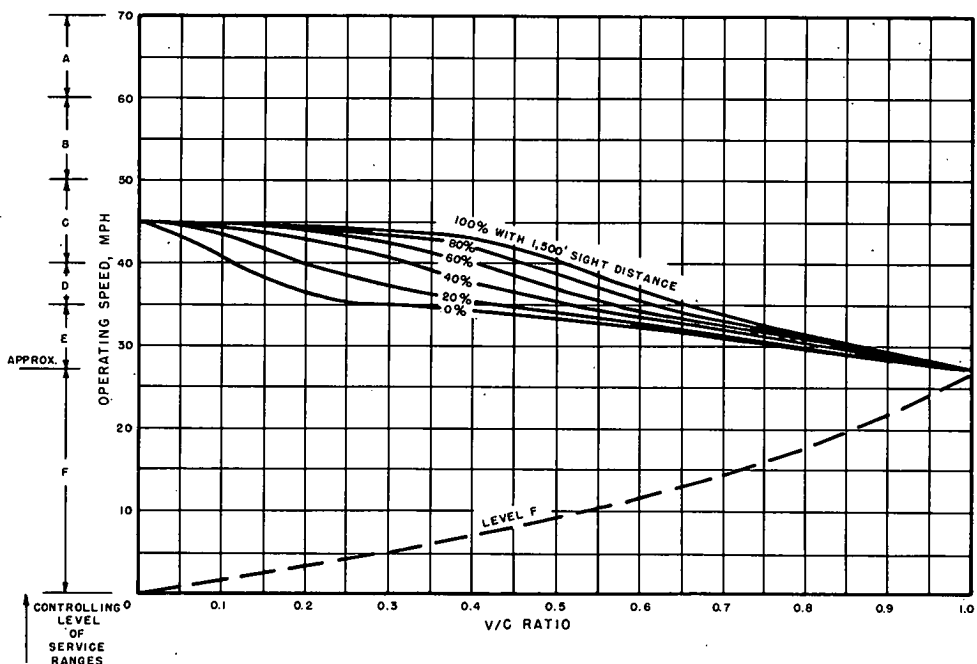
Figure 10.2d. Relationships between v/c ratio and operating speed, overall for both directions of travel, on two-lane rural highways with average highway speed of 45 mph, under uninterrupted flow conditions.
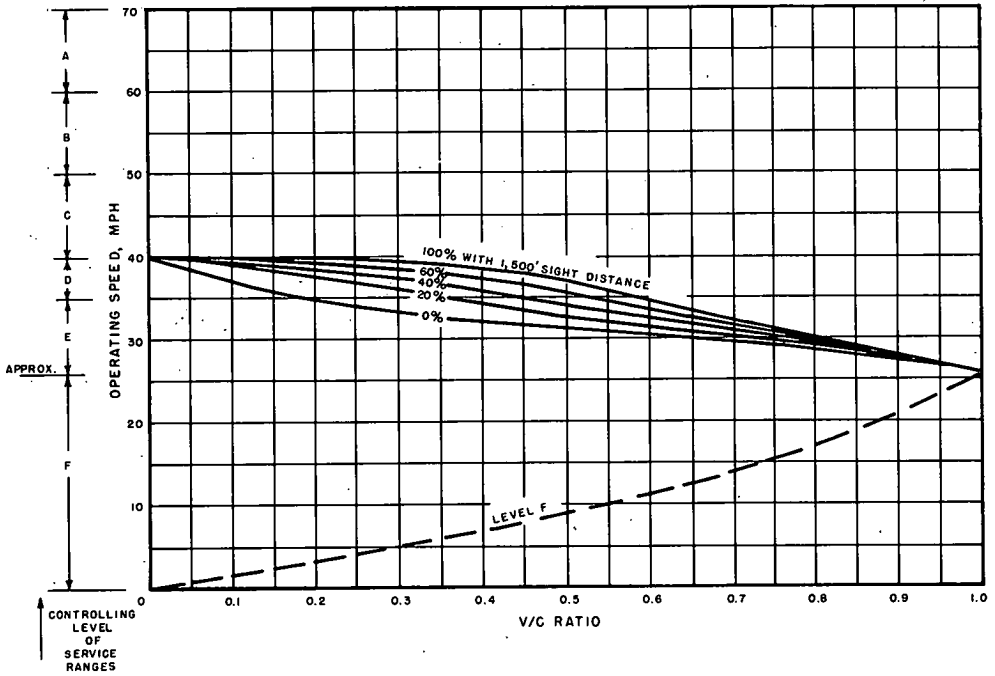
Figure 10.2e. Relationships between v/c ratio and operating speed, overall for both directions of travel, on two-lane rural highways with average highway speed of 40 mph, under uninterrupted flow conditions.
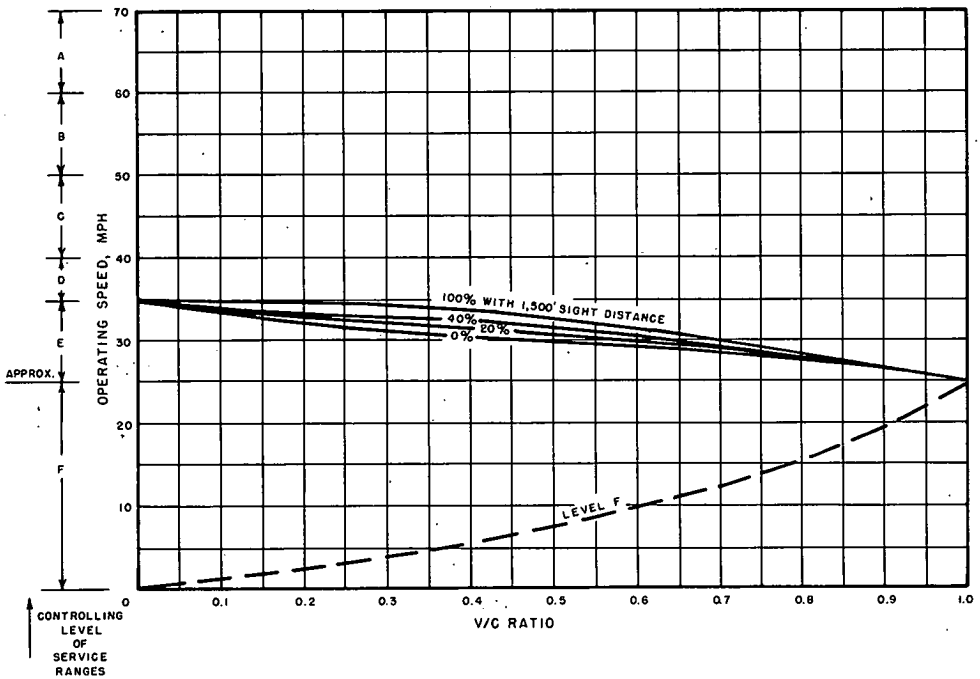
Figure 10.2f. Relationships between v/c ratio and operating speed, overall for both directions of travel, on two-lane rural highways with average highway speed of 35 mph, under uninterrupted flow conditions.

propriate method depends on the particular data already at hand. No matter which method is employed, it is important to check the result by means of Figure 10.2 to confirm that *both* the volume and operating speed criteria for the desired level of service are met, with due consideration for the average highway speed involved.

Computed Directly from Capacity Under Ideal Conditions.—Follow the same procedure as for capacity under ideal conditions, with two exceptions; namely, (1) utilize the adjustments for the appropriate level of service, where different from those for capacity, and (2) apply the appropriate v/c ratio for the level of service desired, the designated percentage of available passing sight distance, and (for typical practical problems) the specified average highway speed.

$$SV = 2,000 \ (v/c) \ W_L \ T_L$$

in which

SV = service volume (mixed vehicles per hour, total for both directions);

v/c = volume to capacity ratio, obtained from Table 10.7 (or Fig. 10.2);

W_L = adjustment for lane width and lateral clearance *at given level of service* from Table 10.8 (shoulder adjustment may be necessary, see Chapter Five); and

T_L = truck factor *at given level of service,* from Table 10.9b for overall highway sections, or Table 10.12 for specific individual grades (intercity buses may have to be considered separately, see text).

Confirm attainment of the desired level of service by checking the resulting operating speed from the appropriate chart in Figure 10.2 for the given average highway speed, to make sure that the level of service requirement has been met.

Computed from Maximum Service Volume for Ideal Conditions.—(Suitable only where alinement is ideal; that is, 70-mph average highway speed and 100 percent passing sight distance.) The procedure is identical to the preceding except that the "maximum service volume" for the level of service desired (from Table 10.7) is used in place of the basic value adjusted by a v/c ratio.

$$SV = MSV \ W_L \ T_L$$

in which MSV is the maximum service volume, in passenger cars per hour, from Table 10.7, and SV, W_L and T_L are as before.

Confirm attainment of the desired level of service, by checking the resulting operating speed from the appropriate chart in Figure 10.2 for the given average highway speed.

CAUTION: Use of this method is not appropriate where restricted average highway speeds or restricted passing sight distances exist, because it does not make use of the v/c ratio in which the influence of these restrictions is incorporated.

Computed from Capacity Under Prevailing Conditions.—Multiply the capacity obtained under prevailing conditions by the appropriate v/c ratio obtained from Table 10.7 for the level of service desired, the appropriate percentage of available passing sight distance, and (for practical applications) the appropriate average highway speed. Also, convert adjustment factors for lane width and lateral clearance and for trucks, both of which are different for levels of service than for capacity.

$$SV = c \ (v/c) \ (W_L/W_c) \ (T_L/T_c)$$

in which c is the capacity (mixed vehicles per hour, total in both directions) as computed for the prevailing conditions, and the other variables are as before.

Confirm attainment of the desired level of service by checking the resulting operating speed from the appropriate chart in Figure 10.2 for the given average highway speed.

Determined from Level of Service Limits. —In the design of a new 2-lane highway, where a specific level of service has been established in advance, service volumes in passenger cars per hour can be read directly from Table 10.7, provided the design is high-type, with near-ideal prevailing conditions. More often than not, however, the design is less than ideal. Where average highway speed, percentage of available passing sight distance, or other conditions are less than ideal, Table 10.7 can be used to determine the limiting v/c ratio. From the controlling

ratio, the service volume can be determined once the capacity is computed. (The appropriate chart in Fig. 10.2 can also be used.)

If a proposed design is already under consideration, the *v/c* ratio obtained can be compared with that for the proposed design, to determine whether or not the design is adequate.

LEVEL OF SERVICE

Determination of the level of service provided by any 2-lane highway section with uninterrupted flow, when accommodating a given demand volume, can be done approximately by use of Table 10.7. However, a refined determination again involves "trial-and-error" procedures, as described earlier for higher types. Here, both the lane width and lateral clearance correction and the truck factor are dependent on the unknown, level of service.

The steps are as follows:

(a) Establish a "base volume" for level of service determination, through the same procedure as described under "Service Volume —Computed Directly from Capacity Under Ideal Conditions," except that no *v/c* ratio is applied.

$$\text{Base volume} = 2{,}000 \, W_L \, T_L$$

(b) Divide the given demand volume by the "base volume" thus computed to obtain the approximate *v/c* ratio. (Conversion of demand volume to equivalent passenger cars is not necessary, because Step (a) has converted the base to mixed traffic.)

(c) Establish the percentage of available passing sight distance greater than 1,500 ft, and the average highway speed.

(d) Reinspect Table 10.7 (or the appropriate chart in Fig. 10.2), if the operating speed was known in advance, to establish the level of service from the controlling factor, operating speed or basic *v/c* ratio.

If the operating speed was not known, enter the appropriate chart in Figure 10.2 for the given average highway speed, computed *v/c* ratio, and the known percentage of passing sight distance, and read the resulting operating speed. Establish the level of service from the controlling factor, operating speed or basic *v/c* ratio,

(e) Recompute, using the revised choice of clearance and truck factors, based on a different assumed level of service, if the initial assumption proves incorrect.

Combined Analysis of Subsections Composing Two-Lane Highway Sections

As with the previously described highway types, if a relatively long section of highway is being examined there will undoubtedly be variations in geometrics and other conditions within the section. Therefore, it may be desirable to establish weighted average overall levels of service. The procedures described previously for ordinary multilane highways remain generally applicable to 2-lane highways as well, with the obvious exception that Table 10.7 and the charts in Figure 10.2 should be used rather than Table 10.1 and Figure 10.1, as there discussed.

In this connection, one important caution is necessary. An overall level of service cannot easily be developed numerically for a highway section composed partly of 2-lane and partly of 4-lane subsections, because of the differing scales involved. Where numerical weighted averages are being developed, therefore, each should be reported separately. Where the graphical method is used, an approximate overall level can be determined. Three-lane sections, to be discussed briefly next, can be combined with 2-lane sections for such purposes.

Typical Problem Solutions—Two-Lane Highways

EXAMPLE 10.3

Problem:

Given:
Rural 2-lane limited-access highway.
12-ft lanes.
10-ft shoulders.
Overall long section, in level terrain.
Ideal alinement; AHS = 70 mph.
100% passing sight distance.
5% trucks.
1% intercity buses.
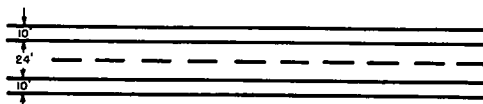
Determine: Service volumes for levels B and E (capacity).

Solution:

Capacity:

$$c = 2,000 \ W_c \ T_c$$

where:

W_c, from Table 10.8 = 1.00, for ideal geometrics.

T_c, from Table 10.9b, for 5% trucks in level terrain = 0.95. (Buses can be neglected; consider as passenger cars.)

$c = 2,000 \times 1.00 \times 0.95 = 1,900$ vph, total for both directions.

Service Volume B:

$$SV_B = 2,000 \ (v/c) \ W_L \ T_L$$

where:

$v/c = 0.45$, from Table 10.7, for level B with ideal geometrics.

$W_L = 1.00$, from Table 10.8.

$T_L = 0.93$, from Table 10.9b, for 5% trucks in level terrain.

$SV_B = 2,000 \times 0.45 \times 1.00 \times 0.93 = 837$ vph, total for both directions.

Operating speed requirement met, by means of the criteria used.

EXAMPLE 10.4

Problem:

Given:

Rural 2-lane highway of low-class design.

10-ft lanes.

No shoulders; obstructions 2 ft from pavement edge on one side; adequate clearance on other side.

Individual grade of 5%, 1½ miles long.

Restricted alinement; AHS = 50 mph.

Passing sight distance = 40%.

7% trucks.

3% intercity buses.

Determine: Service volumes for levels C and E (capacity).

Solution:

Capacity:

$$c = 2,000 \ W_c \ T_c \ B_c$$

where:

W_c, from Table 10.8, for 10-ft lanes with 2-ft clearances on one side = 0.75.

T_c: From Table 10.10, for capacity on 5% grade 1½ mi long, $E_T = 59$.
From Table 10.12, for 7% trucks and $E_T = 59$, $T_c = 0.19$.

B_c: (Substantial bus volume on heavy grade warrants separate consideration.)
From Table 10.11, for 5% grade, $E_B = 2$.
From Table 10.12, for 3% buses and $E_B = 2$, $B_c = 0.97$.

$c = 2,000 \times 0.75 \times 0.19 \times 0.97 = 276$ vph.

Service Volume C:

$$SV_C = 2,000 \ (v/c) \ W_L \ T_L \ B_L$$

where:

v/c (working value), for AHS = 50 mph and 40% passing sight distance = 0.38.

W_L, from Table 10.8 = 0.71 (by interpolation).

T_c: From Table 10.10, $E_T = 51$.
From Table 10.12, $T_c = 0.23$.

B_c: From Table 10.11, $E_B = 3$.
From Table 10.12, $B_c = 0.94$.

$SV_C = 2,000 \times 0.38 \times 0.71 \times 0.23 \times 0.94 = 117$ vph.

Operating speed requirement is met by use of working v/c value.

Note: On this heavy grade, the truck effect is considered to be the same in both directions.

EXAMPLE 10.5

Problem:

Given:

Rural 2-lane highway of intermediate design.

10-ft lanes.

6-ft paved shoulders; no nearby obstructions.
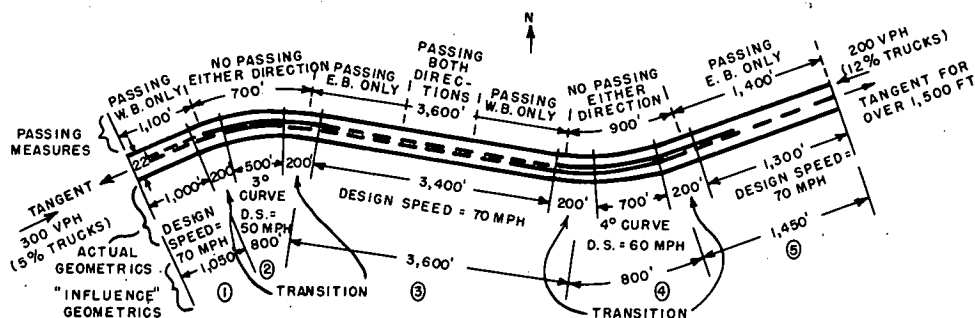
Individual 3% grade, upgrade EB, 7,700 ft long (1.5 mile), starting at left end of sketch.

Alinement as shown in sketch.

Demand volumes: 300 vph, with 5% trucks, EB; 200 vph, with 12% trucks, WB.

Negligible buses.

Determine:

(a) Average highway speed.

(b) Percentage of passing sight distance.

(c) Level of service being provided.

Solution:

(a) Average Highway Speed:

Use the approximate method described in Chapter Five, in which each curve and related transitions are assigned an average influence distance of 800 ft, at the design speed of the curve, regardless of the actual length and degree of curvature. The resulting influence distances are shown in the sketch.

Sub-section	Influence Distance	Design Speed	
1	1,050	(70) =	73,500
2	800	(50) =	40,000
3	3,600	(70) =	252,000
4	800	(60) =	48,000
5	1,450	(70) =	101,500
	7,700		515,000

515,000/7,700 = 67 mph, approx. AHS.

(b) Percentage of Passing Sight Distance:

For the purposes of this problem, zones where no passing is possible in either direction, as shown by the passing distance scale above the sketch, are taken as those portions of the roadway in the sketch which have a double solid centerline. In actual problems, they would be established as those portions not having 1,500-ft passing sight distance in either direction, regardless of markings.

Available 1,500-ft passing sight distance, eastbound:

Subsection 1	None
Subsection 2	None
Subsection 3	3,600 − 1,500 = 2,100
Subsection 4	None
Subsection 5	1,400
	3,500

The available passing sight distance in the opposing direction is nearly the same, as is true in most typical problems; seldom will it justify separate consideration or averaging.

Percentage of passing sight dist. = 3,500/7,700 = 0.45, or 45%.

(c) Level of Service:

Assumption of level must be made before certain adjustment factors can be selected.

Given 500 vehicles with significant

number of trucks on relatively long grade, experience with other problems indicates that service probably will be in level C or D. Assume factors for level D for use in trial computations.

Lanes can be considered as 11 ft wide (from Chapter Five, lanes narrower than 12 ft can be considered 1 ft wider if a paved shoulder 4 ft or more wide is present).

Here, an intermediate grade exists, as do unbalanced percentages of trucks. Data permit refinement of procedure to consider upgrade and downgrade effects separately.

5% of 300 = 15 trucks upgrade.

12% of 200 = 24 trucks downgrade.
$$\overline{500}$$

15/500 = 3% of total volume; upgrade trucks.

24/500 = 4.8 = 5% of total volume; downgrade trucks.

Local observations of downgrade truck speeds, applied to Chap. Five procedures, have determined passenger car equivalent E_T of 10 downgrade.

Base volume $= 2,000\ W_L$
$$\left(\frac{T_{L(up)}\ P_{T(up)} + T_{L(dn)}\ P_{T(dn)}}{P_{T(tot)}}\right)$$

where:

W_L, from Table 10.8, for 11-ft equivalent lanes and adequate clearance, at level D = 0.87.

$T_{L(up)}$: From Table 10.10, for level D, given 3% trucks on 3% grade 1½ mi long, $E_T = 26$.

From Table 10.12, for $E_T = 26$ and 3% trucks, $T_L = 0.57$.

$T_{L(dn)}$: From Table 10.12, for $E_T = 10$ and 5% trucks, $T_L = 0.69$.

Base volume $= 2,000 \times 0.87$
$$\left(\frac{0.57 \times 3 + 0.69 \times 5}{8}\right) = 1,122 \text{ vph.}$$

v/c ratio $= 500/1,122 = 0.45$.

From Table 10.7, for AHS = 67 mph and 45% passing sight distance, service is in level C. Assumption of level was incorrect; recompute for level C for final check.

THREE-LANE HIGHWAYS

Three-lane highways are seldom, if ever, designed and built in the United States today, due to marginal safety considerations and to the fact that they are not a logical step in stage construction directed ultimately toward a 4-lane divided highway. Nevertheless, some are still in operation. Although this chapter does not discuss such highways in detail, this section briefly discusses the capabilities of 3-lane roads.

Basic operational characteristics of 3-lane highways are similar to those of 2-lane roads. For all practical purposes, directional distribution of traffic is not significant in defining operating conditions, although its influence occasionally can be detected. Hence, again, capacity and service volumes are given as totals for both directions.

Little or no recent research has been conducted on the subject of 3-lane roads. Several of the more pertinent conclusions contained in the original edition of this manual are, therefore, repeated here; they are considered to be the most valid available.

1. At any point on a 3-lane highway, relatively few vehicles travel in the center lane. The maximum number that can be in the center lane is about 300 per hour, regardless of the total traffic volume, when up to 70 percent of the total traffic is traveling in one direction.

2. Although there is a marked drop in the average speed of traffic in the outside lanes with an increase in volume, there is no drop in the speeds of vehicles in the center lane.

3. As long as the hourly traffic volume traveling in one direction does not exceed 70 percent of the total traffic, the center lane will be used by vehicles traveling in both directions.

4. The average speed of all vehicles and the capacity of a 3-lane road are slightly higher when the traffic is nearly evenly divided than when two-thirds or more travel in one direction.

· 5. At places where sight distance is restricted, use of the center lane for passing is dangerous; so, in effect, a 3-lane highway will carry only two lanes of traffic at such points.

6. A 3-lane highway having even one restricted sight distance cannot carry more vehicles per hour in one direction than the number that can crowd into one traffic lane —2,000 passenger cars per hour under ideal conditions.

Ideally, then, the capacity of a 3-lane highway under ideal conditions is about 4,000 passenger cars per hour, total for both directions, occurring with operating speeds restricted to about 30 mph. If level of service C is desired, involving operating speeds aver-aging about 40 mph, 2,000 passenger cars per hour, total for both directions, can be carried under ideal conditions, whereas level B operating speeds of about 50 mph can be attained only when volumes do not exceed 1,500 passenger cars per hour, total for both directions under ideal conditions.

Seldom, however, are conditions ideal on 3-lane roads. Generally speaking, the adjustment factors and procedures previously described for 2-lane highways should be applied, as appropriate, to the foregoing ideal capacity and service volume values. This includes consideration of the criteria for percentage of available passing sight distance, because, even though a passing lane exists, it cannot be safely used without adequate sight distance.

Currently, many 3-lane highways in suburban or urban areas are operating under special traffic engineering controls. For instance, the center lane may be utilized reversibly by means of lane control signals, or it may be reserved for left-turn movements only. The general capacity criteria described previously do not apply to such specialized applications; each such specialized case requires local analysis.

URBAN AND SUBURBAN ARTERIALS

The previous sections of this chapter have covered highway facilities operating under essentially rural conditions, with infrequent fixed traffic interruptions and relatively high speeds during free-flow conditions. Fully as important is a large amount of highway mileage located within an essentially urban or suburban environment, where adjacent development has necessitated a closer control of traffic operations through the use of signalization at intersections and/or low speed limits in relation to the quality of alinement. For the purposes of this manual, urban and suburban arterials are defined as major streets and highways outside the central business district having either (1) intersection signalization at an average spacing of 1 mile or less, or (2) speed limits of 35 mph or less due to extensive roadside development. Separate consideration is given later in this chapter to major streets within the central business district.

LEVELS OF SERVICE

Methods of determining and improving the capacity of many different specific bottleneck locations on urban or suburban systems are well known to traffic engineers and have been widely applied. However, the coordinated application of such methods to entire urban routes has not been widely attempted.

Points of traffic interruption, such as intersections, provide logical breakpoints for section analysis. Hence, intersection approach capacity has generally been used as the primary measure of urban capacity.

However, when general service to traffic over the street as a whole is considered, it becomes unrealistic to analyze an urban arterial by means of a series of separate isolated intersection studies alone. Also, such a procedure would not be in harmony with those described in previous sections of this chapter for other roadway types.

In this manual, therefore, the determination of levels of service for both urban and suburban arterials involves relatively long street sections. First, each potential bottleneck location, usually but not necessarily an intersection, is investigated for its effect on and possible control over traffic operations along the entire roadway segment under study. The overall street is then analyzed for overall level of service-capacity relationships. From these two steps, the true nature of the operational conditions encountered by drivers using the street can be determined.

As discussed in Chapter Four, the speed measure used in urban arterial analyses is the average overall travel speed, rather than the operating speed, which has been used under uninterrupted-flow conditions. This modification is necessary because operating speed is difficult to define where a variety of random interruptions exist, whereas the average of the overall speeds of traffic through the complete section is quite easily estimated.

During low to moderate volumes, maximum travel speeds on urban arterials are a function of such factors as speed limits, midblock frictions, intersectional frictions at unsignalized intersections or during green phases at signalized intersections, and the frequency and duration of red phases at signalized intersections together with the number of such intersections. Increasingly as higher volumes develop, intervehicular friction restricts attainable speeds. Quality of alinement has relatively little effect, except in pronounced cases such as at "doglegs," diagonal underpasses, and similar obvious impediments, hence the concept of average highway speed is not applicable. Consequently, speed-volume relationships for urban arterials are significantly different from those on highways having largely uninterrupted flow.

It is not feasible to show any "typical" speed-volume curve for urban arterials, representing actual volumes obtained under ideal conditions, as was done in Chapter Three for the previous highway types described. Where interrupted flow is involved, "ideals" cannot be readily defined, because too many variables are involved and a combination of them which is ideal in one case may be totally out of place in another. Neither can any single speed-v/c ratio curve, or group of curves, represent urban arterial operations all-inclusively, as was done for other highway types; only typical curves can be shown.

Figure 10.3 shows two such representative average overall travel speed-v/c ratio curves, representing perhaps the maximum (Curve I) and typical (Curve II) average overall travel speeds that are likely to be found on typical urban arterials at various volume/capacity ratios. Curve I represents essentially an uninterrupted flow condition, found on unsignalized suburban arterials with 35-mph speed limits or on signalized urban arterials with reasonably good signal progression where stops for red signals are relatively infrequent. Free-flow speed approximates the established speed limit, and average overall travel speeds are only slightly less at low volumes.

Curve II, on the other hand, represents typical interrupted flow conditions. Traffic signals are typically spaced at ½-mile intervals or less and are not interconnected for progression; i.e., vehicle arrivals at any intersection are random or nearly so. Free-flow speed is represented by the speed attained in midblock, in most cases the established speed limit (25 mph in the example represented by Curve II), but seldom can this speed be maintained over an appreciable distance, due to interruptions.

Theoretically, at least, a third curve (for example, Curve III in Figure 10.3) could be drawn to represent a perfect progression, carrying near-capacity volumes in regular platoons at the speed limit, which is equal to the speed of progression, here 30 mph. Progressive signal systems will be discussed in further detail later in this section. Suffice it to say, at this point, that in practice perfect progression is rarely found, although at
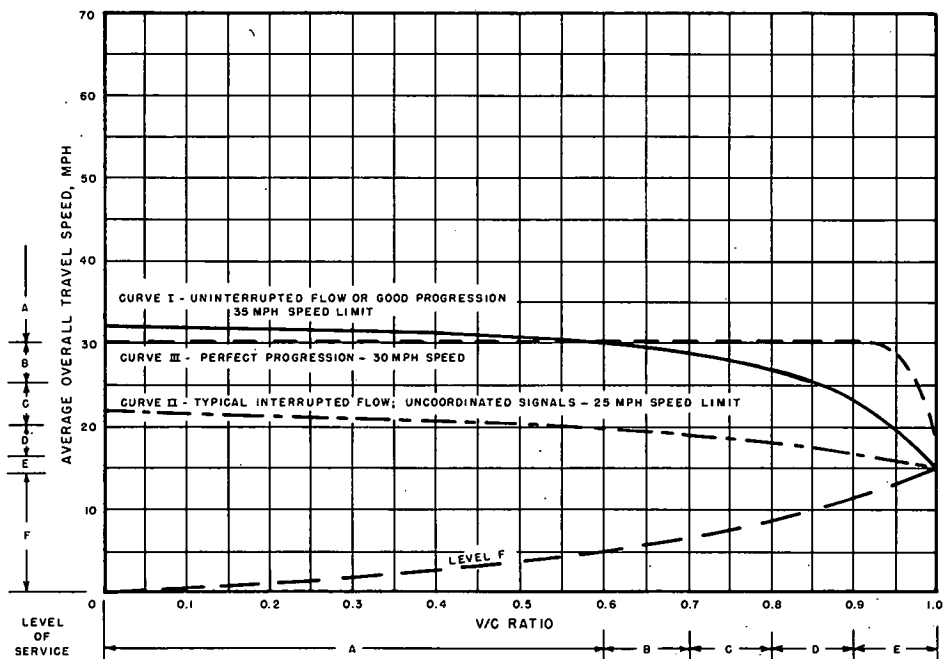
Figure 10.3. Typical relationships between v/c *ratio and average overall travel speed, in one direction of travel, on urban and suburban arterial streets.*

least one progression closely approaching this ideal has been studied in detail (4).

Before describing levels of service, and in particular to avoid misinterpretation of the 1.00 v/c ratio limit in Figure 10.3, it is important to define what is meant by capacity on urban arterials. For uninterrupted flow, infrequently found, capacity is identical in concept, and often in absolute value, to those capacities previously discussed; namely, the maximum number of vehicles past a point in an hour with continuous flow. For interrupted flow, however, capacity normally is not controlled by roadway geometrics, except in special cases such as the presence of a narrow underpass. Rather, it is usually governed by the traffic control features and physical conditions at or near the signalized intersections. Occasionally midblock interruptions and interferences are so significant that they govern instead.

Fundamentally, capacity represents maximum utilization of that portion of the hour during which the highway in question has a green signal indication, or is free of other predictable interruptions. Although a high-type urban street may carry traffic at flow rates approaching uninterrupted flow values while traffic is moving on a green signal indication, there are many periods when traffic does not move or, in the case of progressive systems, when long gaps exist between platoons of vehicles. The capacity in actual vehicles passing per hour, then, is far less than with uninterrupted flow.

Chapter Six has given methods of evaluating individual intersections. When a section of urban arterial of reasonably consistent geometrics contains several signalized intersections, and no radical differences in signal phase timing exist at the several locations, an average condition or level of service (except level E) can be developed to apply to the entire section, as thus defined. However, where maximum volume (level E, ca-

pacity) is being considered, the capacity at the most critical point cannot be exceeded.

Caution must be exercised in defining section lengths and limits, as related to capacity restriction points present within them. Because access is not controlled, the sphere of influence of this critical point is sometimes less than it may at first appear. The significant numbers of access and egress points often found between signals makes invalid the broad assumption that the most critical intersection necessarily controls the overall capabilities of any arbitrarily-chosen street section containing that intersection. Often this is the case, but whether or not such control exists depends on the nature of the local traffic demand. Sometimes, such control exists only to the next points on either side where turning movements are significant, if these turns happen to produce traffic patterns such that the limiting point regularly receives a lighter demand than neighboring subsections. Often, therefore, it will prove more useful to establish the section limits at points of known significant demand change than to place them at known capacity limitation points; the latter procedure tends to hide the nature of the limitation by splitting it into two parts.

Capacities of individual intersection approaches, and hence of midblock locations upstream of them, are determined by means of the methods shown in Chapter Six. In the absence of more detailed local knowledge, those methods assume a load factor of 0.85 and a peak-hour factor of 0.85. Very generally, the capacity of 12-ft lanes on high-type two-way urban arterials, as thus determined, may range from about 1,000 to 1,700 vehicles per lane for each hour of green time, under ideal conditions (no parking, no left or right turns, hence little conflict between vehicles and pedestrians, and no commercial vehicles), depending on city size, location within the city, and total number of lanes. These values should not be used as the basis for computations. The magnitude of the range, even with such important factors as turns and truck percentages removed as variables, serves to show how important it is to consider fully every adjustment shown in the intersection capacity procedures.

Also, before describing urban arterial levels of service, it is important to mention the average overall travel speed limitations which appear in the descriptions. These are largely rationalized values which, in the opinion of the Committee, are most commonly associated with the levels as described. These limits are based primarily on judgment, rather than detailed studies, inasmuch as inadequate field data are available on operation over appreciable lengths of arterials to relate delays and resulting travel times and speeds to the drivers' feelings of satisfaction or dissatisfaction. The Committee considers the listed values and relationships to be representative of the probable opinions of drivers in most cities. It recognizes, however, that local economic problems also must be considered in establishing the level of service that is feasible in any particular city.

Levels of service on urban arterials may be analyzed in a manner similar to that on other highways, here using average overall travel speed and the v/c ratio as criteria. This involves detection and analysis of operating levels of all potential restrictions, and overall analysis of the street. Although bottleneck locations are normally intersection approaches, they may also occur at midblock locations. Levels of service are measured relative to operation of high-type arterials with good progression, as represented by Curve I, Figure 10.3. Hence, many arterials never offer average overall travel speeds high enough to provide the higher levels A and B. On the other hand, in the unusual case of perfect progression, the v/c ratio limitation on level of service is somewhat unrealistic to enforce, because speeds remain consistently high throughout nearly the entire range of volumes.

The scale of levels of service can be approximately related to load factor (used as the measure of level of service for individual intersections), as well as to the likely peak-hour factor. However, it should be realized that, in theory, any peak-hour factor can occur at any given level, because PHF is dependent on the degree of uniformity of demand rather than on its magnitude.

Referring again to Figure 10.3, it can be seen that speed reductions on high-type ur-

ban arterials are slight until the volume/capacity ratio exceeds 0.50. The chance of signal loading (vehicles waiting more than one signal cycle) occurring at any intersection is negligible below this value, with vehicular platoons, when released at a signal, moving out and accelerating with minimum delay and restriction. This could be considered "free flow" for urban arterials, with speeds controlled chiefly by signal progression and regulatory speed limits. Short-term volume fluctuations may reach 1.5 times the hourly rate over several cycles, but have little adverse effect. This assumes free midblock operation, unencumbered by uncontrolled double parking, backups from driveways and parking areas, and similar interferences. On such high-type arterials, at a volume/capacity ratio of 0.60, average overall travel speed should be 30 mph or more, and service volumes at most intersections will approximate those found with a load factor near the limit of the 0.0 range. This is considered the limit of level of service A. Typically, though not necessarily, the peak-hour factor will be about 0.70. (For instance, if demand were uniform throughout the entire peak hour at a flow rate equivalent to this v/c ratio, not quite heavy enough to load any cycle at any intersection, level A could exist with PHF = 1.00. In practice, this combination of conditions is not often found.)

As volumes reach 0.70 of capacity, occasional signal loading may develop. Average overall speeds start dropping due to intersection delay and intervehicular conflicts, but remain at 25 mph or above; delay is not unreasonable. Service volumes at most intersections at this, the limit of level B, will approximate conditions when the load factor is 0.1. The peak-hour factor is likely to be about 0.80 at this level.

Level C extends to service volumes of about 0.80 of capacity. For typical uninterrupted flow on non-signalized streets with commercial development, and for good progressively-signalized operation, average overall travel speeds have dropped to below 80 percent of free-flow speeds, but are at least 20 mph. On streets with typical non-perfect progression, the frequency and duration of loaded signal cycles encountered along the street reaches what is considered a reasonable limit by most drivers. Operating conditions at most intersections approximate a load factor of 0.3; the peak-hour factor is likely to be about 0.85.

Further increases in volume, in level D, begin to tax the capabilities of the street system. Service volumes approach 0.90 of capacity, with average overall travel speeds down to 15 mph. Delays at critical locations, such as crossings of other major arterials, may become extensive, with some vehicles occasionally waiting two or more signal cycles to pass through the intersection. Demand variations are attenuated, with signals in effect storing excess demand. Many of the signalized intersections may reach conditions described by a load factor of 0.7, with the peak-hour factor likely to be about 0.90. These conditions may be tolerable for short periods of time or at occasional bottlenecks, but create unacceptable delay when they exist for a considerable portion of the peak hour along an entire section of street. A properly coordinated progressive signal system will improve operating conditions to a reasonable basis, unless entering volumes at cross streets become great enough to break down the progression.

At capacity, operations on most signalized streets have similar characteristics, regardless of the type of signalization, uncoordinated or a typical progression, because traffic flow has become too unstable for any predetermined signal timing to be consistently correct. Average overall travel speed is variable, but in the area of 15 mph, intersections along the street operate at a load factor in the range between 0.7 and 1.0, and the peak-hour factor is likely to be about 0.95. Continuous backup occurs on the approaches to most intersections, with traffic flows determined by the maximum discharge rates at each intersection. Traffic entering from cross streets supplies enough extra demand to keep most approaches loaded. Traffic seeking to enter or cross from driveways or minor streets can enter only when traffic is stopped upstream at a signal, and even then the maneuver may be difficult.

Under typical interrupted flow operation on signalized urban arterials, the condition

of forced flow (level F) is reached somewhat more gradually than under uninterrupted flow. Flow interruptions are regularly induced at traffic signals throughout the length of the arterial, which in turn meter the traffic into the next downstream section. Storage of excess demand over capacity is therefore distributed throughout the section, as an inherent condition producing capacity. This storage gradually increases, but forced flow is reached on signalized arterials only when the downstream section cannot accommodate the vehicles discharged by a signal, and vehicular backups from one signal ex-

tend back through an upstream signalized intersection and its approaches. This upstream signal and approaches are then operating under forced flow, with resulting lowered volumes at a decreased level of service to the motorist.

Table 10.13 presents the foregoing relationships in consolidated form.

For the unusual case of urban uninterrupted flow at capacity, conditions are quite similar to those found on rural facilities also at capacity, with intermittent stoppages, low speeds, and possible breakdown to forced flow.

TABLE 10.13—LEVELS OF SERVICE FOR URBAN AND SUBURBAN ARTERIAL STREETS

LEVEL OF SERVICE	TRAFFIC FLOW CONDITIONS (TYPICAL APPROXIMATIONS, NOT RIGID CRITERIA)				SERVICE VOLUME/ CAPACITY RATIO[a,c]
	DESCRIPTION	AVERAGE[a] OVERALL TRAVEL SPEED (MPH)	LOAD[a] FACTOR	LIKELY PEAK-HOUR FACTOR[b]	
A	Free flow (relatively)	≥30	0.0	≤0.70	≤0.60 (0.80)
B	Stable flow (slight delay)	≥25	≤0.1	≤0.80	≤0.70 (0.85)
C	Stable flow (acceptable delay)	≥20	≤0.3	≤0.85	≤0.80 (0.90)
D	Approaching unstable flow (tolerable delay)	≥15	≤0.7	≤0.90	≤0.90 (0.95)
E[e]	Unstable flow (congestion; intolerable delay)	Approx. 15	≤1.0 (0.85 typical)[d]	≤0.95	≤1.00
F	Forced flow (jammed)	<15	(Not meaningful)	(Not meaningful)	(Not meaningful)[f]

[a] Operating speed and v/c ratio are independent measures of level of service; both limits should be satisfied in any determination of levels, with due consideration given to the fact that they are largely rationalizations. Load factor, a measure of individual intersection level of service, can be used as a supplemental criterion where necessary.
[b] This is the peak-hour factor commonly associated with the specified conditions; in practice, considerable variation is possible.
[c] Values in parenthesis refer to near-perfect progression.
[d] Load factor of 1.0 is infrequently found, even under capacity operation, due to inherent fluctuations in traffic flow.
[e] Capacity.
[f] Demand volume/capacity ratio may well exceed 1.00, indicating overloading.

CRITICAL ELEMENTS REQUIRING CONSIDERATION

On an urban arterial street, the type of operation provided (signalized, or signalized with progression, and two-way vs one-way) must be considered. In addition, a variety of potential impediments to free flow of traffic exist, resulting primarily from the needs to serve adjacent land uses and to cross other traffic flows frequently.

Signalization

As previously discussed, signalized intersections are such important restrictions that most of Chapter Six is devoted to them. They must be considered as fundamental elements in any determination of the capacity of an arterial section, using the methods described in that chapter. In most cases, further direct computational consideration of the several factors considered in the intersection procedures is not necessary to determine their influence on midblock operation. However, judgment should be exercised. Where, for instance, a heavy grade exists in midblock, further consideration of trucks might be called for.

Signalization with Progression

As mentioned earlier in this Chapter and in Chapters Six and Nine, a perfect or near-perfect signal progression, infrequently obtained, is a special case which can produce quite different operating characteristics. Near-perfect progression can be obtained only in one direction at a time on two-way streets, except in very rare instances. In essence, the fundamental difference in the perfect case is that no vehicles ever stop due to a red signal indication, thus permitting flow rates up to 2,000 passenger cars per hour of green. This type of operation can be attained at high volumes only if the following conditions can be established: (1) there are relatively few turning movements (and those few onto the street largely balance those few off); (2) the demand per cycle can be held slightly under the capacity per cycle to allow a little slack for flexibility within the traffic platoons, and (3) midblock frictional elements are largely absent so that, for all practical purposes, limited-access

operation can be attained. Such high-volume operation is always in delicate balance, however, and is subject to total breakdown whenever any abnormality in the traffic flow develops. A load factor of 0.95 and a peak-hour factor of 0.95 may be suitable for intersection capacity computations under these conditions. Here, the LF = 0.95 has a special meaning, indicating that nearly all cycles were almost, but not quite, fully utilized. This interpretation applies to perfect progression only. In this connection, reference to Curve III, Figure 10.3 will demonstrate the incongruity involved in enforcing the general v/c limits rigidly, under progressive operation. For example, speed stays within level A limits for such a great percentage of the total range, as shown in the figure, that it is unlikely that drivers would find volumes objectionable until a v/c ratio of possibly 0.75 to 0.80 was reached.

Widening of intersection approaches and exits, to provide more lanes for through traffic there than in midblock, is a procedure sometimes followed to offset capacity loss due to "getaway" delays at intersections with ordinary signalization. Where progression is being maintained successfully, such lanes, if provided, are likely to be used only by turning traffic. Because the traffic moves in platoons, with few vehicles having to stop, drivers would seldom find need to move into the extra lanes and would find it difficult to merge back into the moving platoon if they did move out of it. Nevertheless, such lanes may well be a desirable "safety factor" in that they permit turning vehicles to move out of the through flow before slowing down, and stand ready for use to maintain capacity (if not level of service) whenever breakdown of the progressive flow occurs.

In those more usual cases where effective but not perfect progression exists, a certain degree of stop-and-start operation remains but delays are much reduced. Capacity is increased only slightly, if at all, above its level without progression, but travel time through the section is greatly reduced. That is, progression is highly attractive to individual drivers using the route, providing a better overall level of service at any given volume below capacity, in terms of average overall travel speed. However, there may be

little benefit in terms of additional vehicles moved.

One-Way vs Two-Way Operation

The subject of the relative efficiencies of one-way and two-way operation of urban streets has elicited much discussion in recent years, partly because, although early criteria showed substantial benefits in one-way over two-way operation, later interim intersection capacity criteria of the late 1950's showed somewhat contradictory relationships. Various "before" and "after" study results have also shown differing results. In this section, an attempt is made to reconcile these differences.

One-way operation of a given street width is generally more efficient than two-way operation of the same width, in terms of actual vehicles carried per hour, judging by study data currently available (primarily the intersection capacity data described in Chapter Six). However, the degree of one-way superiority varies considerably, depending on the particular situation under study. Under many conditions one-way operation is shown by the intersection capacity curves to be markedly superior to two-way, whereas in other cases little difference is shown.

One exception exists to the foregoing general rule; this involves relatively narrow streets without pavement markings, having parking on both sides and space between just sufficient for two moving lanes of traffic. On such streets there is evidence that somewhat more vehicles are carried with two-way operation than with one-way. (The flow per moving lane is less under two-way operation, but the total flow is more.) Apparently, under such one-way operation, drivers show a tendency to queue up into one lane, rather than accept a "tight" 2-lane flow, whereas with two-way operation, two-lane flow continues because no such choice is available.

Care must be exercised in comparing one-way vs two-way operation. Any meaningful comparison generally requires analysis of the complete "after" system of streets, as compared to the complete "before"; a direct "before-and-after" study of an individual street is difficult and seldom fully valid. For instance, in the simple case of two, two-way streets converted to a one-way pair, or "couplet," an accurate comparison must analyze the overall traffic-carrying capabilities in both directions "after" with those "before." Seldom does a two-way street attain full utilization in both directions simultaneously. Therefore, it would be incorrect to make a comparison of an individual street's operation during a given peak period under such unbalanced utilization "before" with full utilization as a one-way street "after," without consideration of the other half of the pair which frequently would be well below capacity at the same time.

Beyond this point, the conditions which are required for valid comparisons depend on the purpose of the comparison.

For research purposes, valid comparisons further require that the following conditions be equivalent "before" and "after": (1) traffic demand and composition (sufficient to make full use of all lanes simultaneously); (2) pavement condition; (3) parking controls; (4) application of traffic engineering devices; (5) environmental characteristics; and (6) overall turning movements in the system. For some research purposes, equal lane widths may also be required.

Often, in practice, these conditions are not maintained equal during conversion from two-way to one-way operation. Instead, the tendency is to convert ordinary "as is" two-way streets to one-way operation mainly in conjunction with broad upgrading programs which include modernized signalization, new signs and markings, more rigid parking controls, reconstruction, and sometimes revised traffic patterns which route more traffic to the upgraded streets. "Before" and "after" studies of such conversions have great value in showing the overall worth of the improvements. However, if they fail to indicate that other elements were involved in addition to the change to one-way operation, the comparison can be misleading from a capacity and service volume standpoint. Similarly, many reported studies of early one-way operation very likely came from more upgraded sites than did typical two-way studies. Hence, past studies may well have indicated somewhat greater superiority of one-way over two-way operation than is actually the case, "all other things equal."

The foregoing is not meant to detract from the value of one-way operation, but only to draw attention to potential inconsistencies involved in its consideration. On the contrary, it should be stressed that even in those occasional cases where the charts indicate that one-way is little or no better than two-way *on a per-hour-of-green basis,* one-way operation will generally move substantially more vehicles *per actual clock hour* at the same level of service, particularly if complete grids of streets are involved, or the same number of vehicles at a better level of service. This is so because simple two-phase signalization usually is adequate, thus avoiding the time losses due to the extra phases, including extra yellow and/or clearance periods characteristic of the multiphase signalization often required with two-way operation. Another benefit of one-way operation is that signal progression is easier to establish.

Although one-way operation generally is highly desirable, much can be done to upgrade two-way operation to more closely approximate one-way capabilities, where conversion to one-way does not appear to be feasible.

Other Interruptions and Interferences

Along most arterials there exists a variety of other factors which sometimes impede the smooth flow of traffic. Several studies have investigated this broad subject area. In particular, the "Wisconsin Avenue Study" in Washington, D.C., (5) was a comprehensive investigation of urban arterial operating problems. That study report serves as a guidebook in this area.

Problem elements likely to be encountered include the following:

1. Unsignalized intersections.
2. Midblock driveways and related turning movements.
3. Curb parking in midblock.
4. Offstreet parking in midblock.
5. Inadequate signs and markings.
6. Lack of channelization.
7. Restricted lateral clearances.
8. Pedestrian interferences.
9. Transit operations.
10. Non-enforcement of regulations.

Insufficient data are available to attempt to develop detailed correction factors for these various elements. In fact, there is serious doubt that meaningful individual factors could be developed, because these impediments are generally closely interrelated, functioning together as a "team." One major restriction shrouds the effect of the others; correction of this major one may be of little benefit by itself, merely exposing another.

Instead, general problem areas first must be identified by both type and location through on-site investigations. Then decisions must be made as to whether or not certain features of these areas are significantly restricting capacity or level of service, through adaptation of the available analysis procedures and criteria previously established to the problem at hand. In the case of frequently used midblock driveways and alleys, for example, it might prove best to analyze them as if they were signalized intersections, with assumed cycle times. Finally, administrative decisions become necessary regarding the feasibility of eliminating the restrictions.

With reference to turning movements, it is important to remember that any "blanket" prohibition of all left turns, in midblock as well as at intersections, without careful consideration of alternate paths available to fulfill the desired movements, can sometimes defeat its own purpose, due to resulting erratic driving (while searching for alternates) and longer routes. Where periodic gaps appear in the opposing traffic stream due to signalization upstream, an effective compromise often consists of protected left-turn bays or a continuous left-turn lane; drivers can safely wait until a break occurs in opposing traffic.

Where transit routes are operated on urban arterial streets, they too have an effect on the capacity of the street. Insofar as capacity calculations are concerned, their effects are incorporated into the intersection capacity determination procedures described in Chapter Six. No additional adjustments need be made for local bus transit, although the overall subject of transit operations is further discussed in Chapter Eleven.

COMPUTATION PROCEDURES FOR URBAN ARTERIAL STREETS

Determination of the overall level of service provided by very high-type urban arterials may be possible by the methods in Chapter Nine for expressways, if the conditions there specified are met. Procedures for determining overall levels of service for more ordinary sections of urban arterials of appreciable length, not including downtown streets, are different and less specific than those for the highway types previously discussed. They make use of certain of the procedures previously described, but also require application of good judgment. In particular, basic building blocks in the form of uniform subsections are largely lacking, or at least are masked by point restrictions within them. Rigid all-inclusive criteria, therefore, cannot be presented regarding speed-v/c ratio relationships, making careful consideration of findings necessary for meaningful interpretations.

Basic Components

Usually, it is not feasible to divide an urban arterial into several subsections for analysis, in the same manner as is done on rural highways; too many variables are involved. In practice, in the past, individual intersection capacities and service volumes were the primary measures used in considering urban street capacities. Gradually, however, more thought has been given to the entire street as a unit; this overall approach is used here.

For the purposes of this manual, therefore, in any detailed urban arterial study the capabilities of each important intersection are determined by the methods of Chapter Six, whereas those of significant midblock restrictions are estimated by adaptations of basic methods. Also, average overall travel speeds through the section are determined by test runs or estimated by comparison with known speeds on similar type streets. Thus, point capacities and service volumes, and average overall travel speeds, are the primary building blocks available.

Overall Analysis of Urban Arterial Streets

Curve I of Figure 10.3, as previously mentioned, represents the relationship between the service or demand volume/capacity ratio and average overall travel speed which has been accepted as representative of conditions over significant distances on typical high-type arterials. Table 10.13, which summarizes the limiting values of the v/c ratio and the average overall travel speed which define levels of service, is based on this curve. Although either can be used as a guide for computations applying to such streets, they should not be considered fundamental for all urban arterial conditions in the same sense as similar criteria for uninterrupted flow. Instead, each user may find it necessary to develop local curves similar to Curve II to represent typical conditions in his own locality.

The following general steps are suggested.

CAPACITY

1. Make an overall review of the street section under study, and establish the elements that may influence the capacity. Usually, these elements will include: all signalized intersections; midblock locations restricted either by geometrics, traffic interferences such as at entrances or exits, or special traffic controls; and, occasionally, a relatively uninterrupted-flow section existing between two signals spaced more than 1 mile apart. (It should be noted that sometimes signalized intersections are present which do not influence capacity, such as those provided to permit pedestrians or minor side street traffic to cross a heavy flow occasionally. Such signals often have such short green periods for the cross movement that they do not appreciably affect capacity, although they influence level of service. On the other hand, some signals handling heavy pedestrian volumes may be capacity bottlenecks because timing is governed by pedestrian, rather than vehicular, needs.)

2. Compute the capacities of significant intersection approaches by the methods given in Chapter Six, and of any uninterrupted-flow sections by the methods described earlier in this chapter. Analyze each significant midblock restriction as a special case. No specific procedures can be described for these analyses; procedures already covered in this manual must be adapted to each particular case. Very often, intersection capacity determination procedures can

Special signalized "jug-handle" turning lanes minimize intersectional friction on this suburban arterial street. Note frontage road on right minimizing land use interference and conflicts from land use on left.

be so adapted to represent the influences of lost time and/or restricted width, or the geometric factors covered in Chapter Five may be applicable.

3. Interpret the results of the foregoing analyses to establish (a) obvious bottleneck locations having capacities considerably less than the street as a whole, and (b) a capacity level for the remainder of the section, exclusive of the bottlenecks, governed by the minimum capacity of the remaining sections.

4. Desirably, make an effort to increase the capacity of the bottlenecks to the con-

trolling value of the remainder of the section. If this is not possible, these points will govern capacity, at least in their immediate area. However, because many entrance and exit points exist, it is possible that other less restricted sections of the street will actually handle more traffic than can pass through the bottleneck. This point is discussed further under "Level of Service" below. Hence, the controlling value will apply elsewhere.

SERVICE VOLUMES

Follow the same steps as just described for capacity determination, substituting service volume determination criteria for capacity criteria, up to the point of interpretation of the results.

In interpreting the results, it should be remembered that a highway element's inability to meet a service volume criterion is not as critical as its inability to meet the capacity level of the section as a whole. To a degree it will still be a bottleneck, providing poorer service than the remainder of the section, but it usually can carry the load.

LEVEL OF SERVICE

In any typical problem, the desirable goal is determination of an overall level of service for an entire arterial or at least a major section thereof. Analysis, however, generally requires subdivision; the procedure is as follows:

1. Make an initial overall review of the street, to determine those points at which traffic composition quite obviously changes markedly due to turning movements at cross streets, ramps, or other entrances and exits. These points should be established as section limits for the purposes of further analysis. (Where traffic composition does not change widely along an arterial, but where intersection problems are significant, it is sometimes wise to establish midblock control points, rather than points at intersections, so that all of the problems connected with any one intersection will be included within the same control section.)

2. Through use of the procedures described in Chapter Six, determine the capaci-

ties of all intersections and other elements within the section which offer any possibility of influencing the operation of the arterial, as just described under "Capacity." As done there, separate obviously abnormal restrictions, and determine the controlling capacity of the remainder of the section.

3. Determine whether or not the overall demand volume exceeds the controlling capacity of the section. Where this capacity is not exceeded, check further to determine whether any of the abnormal locations separated for individual analysis have limiting capacities less than the demand volume.

4. Where Step 3 does not produce a limiting capacity, divide the demand volume by the controlling capacity to obtain the average v/c ratio for the section. From Figure 10.3, or an equivalent chart developed locally to better fit prevailing conditions, obtain the typical average overall speed for the basic type of street involved and determine the corresponding overall general level of service from the figure or Table 10.13 or local equivalent.

Where abnormal restrictions are present, though not capacity limitations, consider each in sufficient detail to establish a point level of service. Often this is done by the methods of Chapter Six, directly in the case of intersections or adapted for other interruptions, but through-roadway methods sometimes are more suitable. Interpret those point levels in terms of the number of such restrictions, local acceptability in relation to the controlling level obtained over the remainder of the section, and the feasibility of modifications to raise the level provided. Then establish a final level of service for the section by approximate weighting in terms of restriction influence distances.

5. Where Step 3 produces a limiting capacity, make more detailed analyses of the limiting point to ascertain the extent of its influence; that is, determine whether it has only local effect due to turning movements occurring upstream and downstream, or whether it creates stop-and-go operation (level F) upstream while metering traffic downstream at a tolerable level. Assign overall level of service accordingly.

Example 10.6, which follows in the next section, is a typical problem demonstrating use of the methods.

Conversely, to make an approximate determination of the service volume provided by a section of urban arterial, given the level of service or average overall speed desired, enter Figure 10.3 or locally-prepared equivalent for the basic type of street operation involved, and read the v/c ratio. Then, apply this to the controlling capacity of the section, determined as just described, to determine the service volume (or demand that can be handled on this street at this level).

It should be emphasized that all of the foregoing procedures for urban arterials are approximate, suitable only as guidelines for general application. Where an arterial is being examined prior to costly improvements, far more detailed procedures should be employed to consider carefully the many potential problems existing at critical areas.

Typical Problem Solutions—Urban Arterial Streets

EXAMPLE 10.6

Problem:

Given:

 Urban signalized two-way arterial street segment.

 Widths as shown in sketch.

 Curbed (6-in. curbs).

 Level.

 No parking.

 3% trucks throughout.

 30 local buses per hour; stop as shown in sketch.

 Outlying business district.

 City size = 500,000 population.

 PHF = 0.85.

 Pedestrian interference negligible.

 Intersection and turning movement characteristics as shown.

 Timed runs indicate average overall travel speed of 19 mph.

 Eastbound flow under consideration; demand volumes as shown.

Determine:

 (a) Through level of service indicated by average overall travel speed.

 (b) Level of service indicated by in-

CYCLE = 70 SEC
GREEN = 35 SEC

MIDBLOCK TURNS WAITING
TO ENTER PARKING LOTS
BLOCK LEFT AND RIGHT
LANES CONTINUOUSLY, FOR
ALL PRACTICAL PURPOSES.
(DELAYS DUE TO INTERNAL
CONGESTION IN PARKING LOTS,
PRIMARILY.) CENTER LANE
BLOCKED BY OPPOSING LEFT
TURNS FOR 30% OF HOUR.

CYCLE = 70 SEC
GREEN = 25 SEC
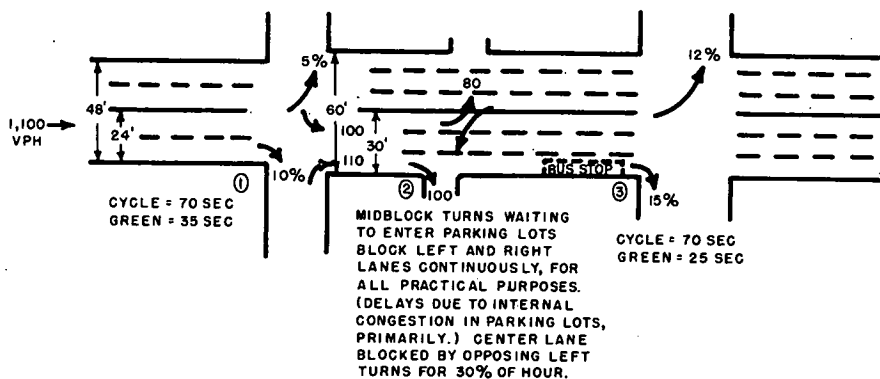
tersection and midblock restriction performance.

(c) Evaluate results.

Solution:

(a) Through level of service indicated by average overall travel speed:

From Table 10.13, for 19 mph, level of service is D, but not far from level C limit.

(b) Level of service as controlled by restrictive elements:

Review indicates that intersection 1, driveway entrance area and intersection 3 are the main controlling elements.

Intersection 1

Fig. 6.8 applies.

Determination of chart volume:

From Fig. 6.8, for 500,000 population and PHF = 0.85, factor = 1.06.

From Fig. 6.8, for outlying business district, factor = 1.25.

From Table 6.4, for 10% right turns, factor = 1.00.

From Table 6.5, for 5% left turns, factor = 1.05.

Because there is no bus stop, local buses can be considered as trucks. Therefore 30/1,100 = 2.7% buses, say 3%.

From Table 6.6, for 3% + 3% = 6% trucks, factor = 0.99.

G/C ratio = 35/70 = 0.50.

Chart volume =

$$\frac{1,100}{1.06 \times 1.25 \times 1.00 \times 1.05 \times 0.99 \times 0.50}$$

= 1,597 vphg, under base conditions.

Intercept of 1,597 vphg and 24-ft width shows LF = 0.15; intersection level of service C indicated.

Capacity, from Fig. 6.8 = 2,100 vphg; $v/c = 1,597/2,100 = 0.76$, in level C, from Table 10.13.

Driveway entrance area 2

Approximate method of handling must be developed.

Demand volume = 1,100 − 1,100 (0.10 + 0.05) + 100 + 110 = 1,145 vph.

Because opposing turns obstruct the through flow, flow through the block is stop and go, as it would be through a signalized intersection.

Assume signalized intersection with no turns, 10-ft approach width, no parking, and 70% green time (100 − 30).

Chart volume =

$$\frac{1,145}{1.06 \times 1.25 \times \underset{\text{R.T.}}{1.20} \times \underset{\text{L.T.}}{1.30} \times 0.99 \times 0.70}$$

= 800 vphg, under base conditions.

From Fig. 6.8, intercept of 800 vphg and 10-ft width shows LF=0.9, within level of service E, and at or near capacity.

Intersection 3

Demand volume=1,145−100−80 =965 vph.

Factors:

Population and PHF (Fig. 6.8) = 1.06.

Outlying business district (Fig. 6.8) = 1.25.

15% right turns (Table 6.4) =0.99.

12% left turns (Table 6.5) = 0.98.

3% trucks (Table 6.6) =1.02.

30 buses per hr, near-side stop (Fig. 6.11) =0.91.

G/C ratio=25/70=0.36.

Chart volume=

$$\frac{965}{1.06 \times 1.25 \times 0.99 \times 0.98 \times 1.02 \times 0.91 \times 0.36}$$

=2,246 vphg, under base conditions.

Intercept of 2,246 vphg and 30-ft width shows LF=0.5; center of intersection level of service D indicated.

Capacity, from Fig. 6.8 = 2,700 vphg; v/c=2,246/2,700=0.83, in level D, from Table 10.13.

(c) Conclusions: The overall street is moderately heavily utilized. Overall, it is near the start of level of service D, and the individual intersections are in levels C and D, respectively. (For urban arterials, through and intersection levels of service would be expected to be nearly alike, by definition.)

However, the midblock pair of driveways is a serious "spot" interference. This location, rather than the signalized intersections, controls the capacity of this portion of the street.

EXAMPLE 10.7

Problem:

Given:

Urban one-way arterial street, with near-perfect signal progression.

44 ft wide, with 4 11-ft lanes; no parking.

Curbed (6-in. curbs).

Level.

Negligible trucks.

No local buses.

Fringe area.

City size=1,000,000 population.

PHF=0.95.

Pedestrian interference negligible.
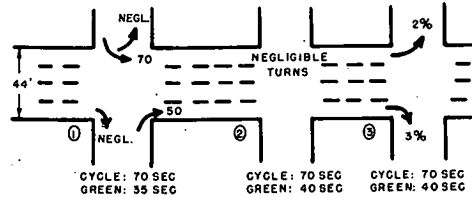
Determine:

(a) Capacity of the arterial.

(b) Capability of absorbing the turns into the street at intersection 1.

(c) Relationship to freeway capacity.

Solution:

(a) Capacities:

Figure 6.5 applies.

For perfect progression, use PHF= 0.95 and LF=0.95 (see text).

Intersection 1

For 44-ft width and LF=0.95, chart volume=4,300 vphg.

Adjustments:

1,000,000 pop. and PHF of 0.95 (Fig. 6.5) =1.22.

Fringe area (Fig. 6.5) =1.10.

G/C ratio=35/70=0.50.

Right turns (negligible) (Table 6.4) =1.00.

Left turns (negligible) (Table 6.4) =1.00.

Trucks negligible (Table 6.6) = 1.05.

No local buses=1.00.

Capacity = $4{,}300 \times 1.22 \times 1.10 \times 0.50 \times 1.00 \times 1.00 \times 1.05 \times 1.00 = 3{,}030$ vph.

Intersection 2

Same adjustments as for intersection 1, except:

G/C ratio = $40/70 = 0.57$.

Capacity = $3{,}030 \times 0.57/0.50 = 3{,}455$ vph.

Intersection 3

Same adjustments as for intersection 2, since on streets of this width turning movement adjustment is 1.00 regardless of percentage.

Capacity = $3{,}455$ vph.

Note: Although there is no change in turning movement adjustment, substantial turns would interfere with traffic sufficiently to break down the progression.

(b) Evaluation:

At intersection 1 capacity is 3,030 vph.

At intersections 2 and 3 capacity is 3,455 vph, due to increased green time.

$3{,}455 - 3{,}030 = 425$ vph.

Street can therefore accommodate the 120 vehicles entering at intersection 1, an average of between 2 and 3 vehicles per cycle, provided the additional 5 sec of green time at intersections 2 and 3 is added at the start of the phase so that the added vehicles (which will be stopped at intersection 2 by a red indication) can accelerate and clear before the next progressive platoon arrives.

(c) Comparison with freeway capacity:

Capacity of 44-ft freeway roadway, with other conditions ideal:

From Table 9.1, ideal capacity = 8,000 vph.

From Table 9.2, width adjustment = 0.96.

$8{,}000 \times 0.96 = 7{,}680$ vph.

For 50% "go" time, equivalent to urban green time, $7{,}680 \times 0.50 = 3{,}840$ vph.

$3{,}030/3{,}840 = 0.79$.

Thus, this progressive system, on a per-hour-of-green basis, has about 80% of the traffic-carrying capability of an equivalent freeway roadway.

DOWNTOWN STREETS

GENERAL CONSIDERATIONS

In the previous sections of this chapter traffic operations on highways and streets with progressively greater proportions of service to adjacent properties, in relation to their traffic service function, have been described. Even on urban arterial streets, however, relatively high standards of speed and freedom to maneuver have been established for level of service A.

In the central business district, on the other hand, many important streets have as their primary function service to local businesses, by passenger cars, transit buses, and trucks. Efficient service to through traffic is often of secondary concern, although certain strategically located downtown streets may be converted to an arterial-type operation during the peak commuter hours. Typically, the downtown flow involves a substantial percentage of circulatory, rather than straight-through, movements, and heavy pedestrian volumes conflict with the large number of turning vehicles involved. A considerable number of transit buses and single-unit local-service trucks are present, which, though performing highly essential services, nevertheless restrict smooth flow because of the stop-and-start curb-lane-blocking nature of their operation.

It is not yet feasible to develop charts or curves presenting basic speed-volume relationships for extended sections of downtown streets, for the same reasons mentioned earlier for urban arterials. At present, with the current limited knowledge of the complex relationships that govern downtown traffic flow, it is not possible to develop even

Two-way downtown street, showing application of signalization and pavement laning.

typical speed-v/c ratio relationships. The capacities of apparently similar downtown streets can vary widely, due to differing environmental conditions.

Very simply, many downtown operations would fall in level F if measured against the level of service rating scale for the higher-type urban flows previously described. That is to say, downtown operation in any one block is often influenced by conditions in other nearby blocks, and speeds are sufficiently low that they fall on the lower, or "breakdown," curve of the typical speed-volume relationship; any increase in speed would thus increase volumes carried.

In the opinion of the Committee, it is not realistic to relate downtown street operation, over sections extending for several blocks, to the rating scales for other urban streets. Neither is it considered feasible, in the pres-

ent limited state of knowledge, to provide procedures for determining a level of service, given a demand volume. However, there is value in suggesting at least a rudimentary level of service scale for such streets, against which an existing flow can be compared. This scale, given in Table 10.14, presents the Committee's views regarding the average driver's degree of acceptance of various operating levels. It is based entirely on average overall travel speeds. No attempt is made to relate them to volumes carried, because so many factors and frictions are present. In particular, closely spaced intersections, each accommodating significant vehicular and pedestrian volumes on the cross street, are present. This requires that signals be set to accommodate a greater percentage of cross traffic than is often the case on outlying arterials. Because each inter-

TABLE 10.14—LEVELS OF SERVICE FOR DOWNTOWN STREETS

LEVEL OF SERVICE	TRAFFIC FLOW CONDITIONS (APPROXIMATIONS, NOT RIGID CRITERIA)	
	DESCRIPTION	AVERAGE OVERALL SPEED (MPH)
A	Free flow (relatively; some stops will occur)	≳25
B	Stable flow (delays not unreasonable)	≳20
C	Stable flow (delays significant but acceptable)	≳15
D	Approaching unstable flow (delays tolerable)	≳10
Eª	Unstable flow (congestion not due to back-ups ahead)	Below 10 but moving
F	Forced flow (jammed)	Stop-and-go

ª Level E for the downtown street as a whole cannot be considered as capacity; capacity is governed by that of controlling intersections or other interruptions.

section has its own characteristics, and turns are frequent, each controls only the block immediately upstream, in most cases. Hence, performance over a section including several blocks is almost impossible to specify in the absence of detailed local knowledge.

It is recommended, therefore, that downtown streets be first analyzed intersection by intersection, for capacity or service volume purposes, by means of the methods presented in Chapter Six. Given a knowledge of travel times (and, therefore, of average overall travel speeds) through the section, a general measure of the level of service, as related to

the range in levels typically found in downtown areas, can then be obtained from Table 10.14.

Typical Problem Solutions—Downtown Streets

EXAMPLE 10.8

Problem:

Given:

Downtown street segment 4 blocks long; all intersections signalized. Two-way, with parking as shown. Width 56 ft, curb to curb.
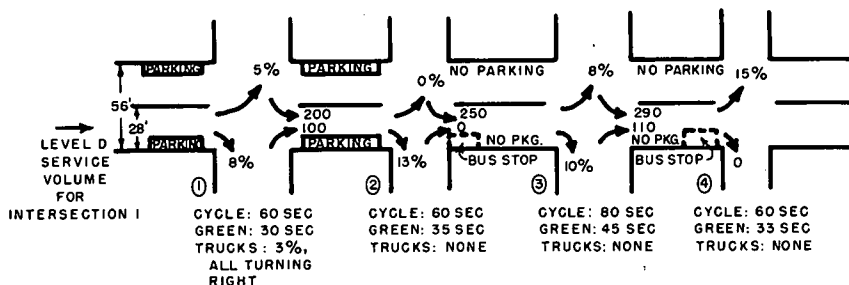
City size = 175,000.

PHF = 0.85.

Intersection and traffic characteristics as shown.

40 buses per hour, making local stops.

No separate turning movement lanes or signal phases.

Eastbound flow under consideration.

Determine:

(a) Approximate through level of service being provided, if timed runs show average overall travel speed of 14 mph.

(b) Intersection service volumes, at intersection level of service equivalent to through level determined above.

(c) Controlling intersection, for traffic demand pattern as shown in Part (c) of solution.

Solution:

(a) The timed runs show an average overall travel speed of 14 mph. From Table 10.14, this is in downtown street level of service D, for through flow.

(b) Intersection Level D Service Volumes:

Intersection 1

Figure 6.9 applies.

For 28-ft width and LF = 0.7, chart volume = 1,550 vphg.

Adjustments:

For 175,000 pop. and PHF of 0.85 (Fig. 6.9) = 0.97.

For downtown (Fig. 6.9) = 1.00.

For G/C ratio of 30/60 = 0.50.

For right turns:

Here, where all trucks turn, special consideration is justified. ⅝ths of the turns are trucks, with pass. car equivalent of at least 2.

⅝ × 1 + ⅜ × 2 = 1 1/8.

Assume turns to be 11% equiv. pass. cars (Table 6.4) = 0.995.

For left turns, 5% (Table 6.5) = 1.05.

40 buses per hour; no stop.

Consider as trucks, with approx. percentage, based on inspection of factors thus far, of 5%.

Trucks, 3% actual plus 5% buses = 8% (Basic adj., Table 6.6) = 0.97.

$SV_D = 1,550 \times 0.97 \times 1.00 \times 0.50 \times 0.995 \times 1.05 \times 0.97 = 760$ vph.

Intersection 2

Fig. 6.9 again applies.

Adjustments:

Pop. and PHF = 0.97.

Downtown = 1.00.

G/C ratio, 35/60 = 0.58.

Right turns, 13% = 0.985.

Left turns, 0% = 1.10.

Trucks, 0% = 1.05.

40 buses, far-side stop (Fig. 6.14) = 1.00 (max).

$SV_D = 1,550 \times 0.97 \times 1.00 \times 0.58 \times 0.985 \times 1.10 \times 1.05 \times 1.00 = 992$ vph.

Intersection 3

Fig. 6.8 applies.

For 28-ft width and LF = 0.7, chart volume = 2,250 vph.

Adjustments:

Pop. and PHF (Fig. 6.8) = 0.97.

Downtown (Fig. 6.8) = 1.00.

G/C ratio, 45/80 = 0.56.

Right turns, 10% = 1.00.

Left turns, 8% = 1.02.

40 buses, no stop = 1.05.

(Inspection shows 40 buses to be probably about 3%, to be considered as trucks)

Trucks, 0% + through buses, 3% = 1.02

$SV_D = 2,250 \times 0.97 \times 1.00 \times 0.56 \times 1.00 \times 1.02 \times 1.05 \times 1.02 = 1,335$ vph.

Intersection 4

Fig. 6.8 again applies.

Adjustments:

Pop. and PHF = 0.97.

Downtown = 1.00.

G/C ratio, 33/60 = 0.55.

Right turns, $0\% = 1.025$.
Left turns, $15\% = 0.95$.
Trucks, $0\% = 1.05$.
40 buses, near-side stop (Fig. 6.11) $= 0.82$.
$SV_D = 2,250 \times 0.97 \times 1.00 \times 0.55 \times 1.025 \times 0.95 \times 1.05 \times 0.82 = 1,006$ vph.

Service volumes for level D:
Int. $1 =$ 760 vph.
Int. $2 =$ 992 vph.
Int. $3 = 1,335$ vph.
Int. $4 = 1,006$ vph.

Intersection 1 appears to be the controlling intersection.

(c) Confirmation of controlling intersection:

Particularly in a downtown area where turns into the street under consideration from cross streets are likely to be quite heavy, a review of the actual demand volume pattern is necessary before final conclusions are drawn regarding the controlling intersection.

Assume the following entering traffic volumes:
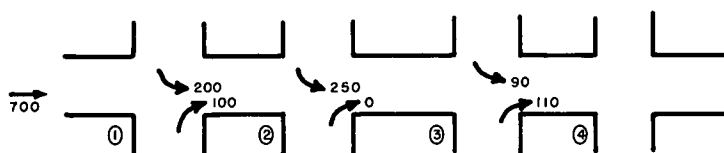
Intersection 1:
Approaching $=$ 700
Leaving to side
$700(0.05 + 0.08)$ $= -91$
Entering from side
$200 + 100$ $= +300$

Intersection 2:
Approaching $=$ $909 < 992$, SV_D, Int. 2; satisfactory.

Leaving to side
$909(0.13)$ $= -118$
Entering from side $=$ 250

Intersection 3:
Approaching $= 1,041 < 1,335$, SV_D, Int. 3; satisfactory.

Leaving to side
$1,041(0.18)$ $= -187$
Entering from side $=$ 200

Intersection 4:
Approaching $= 1,054 > 1,006$, SV_D, Int. 4; not satisfactory.

Conclusion:

Although intersection 1 appeared at first to control, based on service volumes alone, further analysis shows that under the given traffic circulation pattern in this downtown area intersection 4 is actually more critical. It will reach capacity first, producing backups extending to other locations.

This problem demonstrates the fact that, where turning movements are sig-

nificant along either a downtown street or an arterial, an apparent restriction may well be less controlling than it at first appears.

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Four-lane urban arterial flow is provided here by prohibiting parking on one side during peak travel times.

BUS TRANSIT

INTRODUCTION

Metropolitan areas vary greatly in population density and land use; therefore, the means of accommodating commuting travel peaks likewise vary. Table 11.1 indicates the variation in peak-period use of all forms of public transportation entering the central business district in some cities.

Not all forms of public transportation, or "mass transit," are directly involved in highway capacities and levels of service. The term "transit" as used in this chapter refers to the motor bus mode, operating on urban highways and streets. Therefore, the terms "bus," "buses," and "transit" will be used interchangeably.

Inasmuch as the pattern of properly located arterial streets and highways in urban areas generally fits the desire lines of all

forms of travel, particularly as related to central business districts, public transit can do much toward relieving congestion on existing facilities and increasing capacities of old and new highways in terms of total passengers carried (1, 2). Although they require more room per vehicle on the street or highway because of their size and operating characteristics than do private automobiles, transit vehicles carry many more passengers per unit than automobiles and, therefore, reduce the total number of vehicles in the traffic stream.

The "mix" of automobiles and transit vehicles in the traffic stream results from the choice of travel mode by the traveler and, in the case of the bus, a determination by the transit operator of the number of transit vehicles to be scheduled over the artery to handle adequately the persons desiring to travel that way (1). With a knowledge of the composition of the traffic stream, the highway engineer can turn to this manual for guidance in vehicular capacity determinations.

Transit utilizes the highways in several ways. Local street systems allow transit to provide access to residential or commercial areas. Arterials provide both local and express transit services. Combinations of local and arterial streets can afford local-express transit service, or serve as access routes to parking areas near major rail or bus transit stations. Expressways and arterial streets can be used for express transit service either in mixed traffic or in exclusive lanes on the right-of-way. When new arterial highways are constructed, they present the opportunity for improvement of transit service (2).

This transit chapter presents information relating to highway capacity for transit vehicles in mixed traffic. A small amount of information is also presented on the lane

TABLE 11.1—PEAK-PERIOD USE OF PUBLIC TRANSPORTATION ENTERING CENTRAL BUSINESS DISTRICT

METROPOLITAN AREA	YEAR	PERSONS ENTERING CBD BY ALL FORMS OF PUBLIC TRANSPORTATION (%)
Chicago, Ill.	1960	86
New York, N. Y.	1962	85
Newark, N. J.	1960	80
Philadelphia, Pa.	1955	57
Cleveland, Ohio	1960	56
San Francisco, Calif.	1959	46
Los Angeles, Calif.	1960	34
Houston, Tex.	1963	23

Heavy use of an urban intersection by buses operating on both the main roadways and frontage roadways.

Transit operation on frontage roadways.

Multiple use of a near-side bus stop. Buses have completed loading and are leaving on green signal.

capacity when transit vehicles are operating in exclusive lanes. These latter data, however, have not been verified through extensive research or use and are therefore presented for information only.

EFFECT OF TRANSIT ON HIGHWAY CAPACITY

Transit generally moves on the highway as a component of mixed traffic using arterials, local streets and access connections in common with automobile and truck traffic.

·TABLE 11.2—OBSERVED PEAK-HOUR VOLUMES OF LOCAL BUSES ON CITY · STREETS WITH PARKING PROHIBITED[a]

CITY	FACILITY	LENGTH OF SEC-TION (MI)	TRAFFIC LANES, ONE DIREC-TION	TRANSIT ROUTES USING STREET	SERVICE STOPS IN SEC-TION	BUS MOVEMENT		AUTO-MO-BILES
						NO. OF BUSES	HEAD-WAY (MIN)	
New York	Hillside Ave.	0.6	3	9	6	150	0.4	—
San Francisco	Market St.	1.1	3	8	8	130	0.5	730
Cleveland	Euclid Ave.	1.0	3	7	10	90	0.7	860
Chicago	Michigan Ave.	0.3	3	9	4	175	0.3	1,416
Baltimore	Baltimore St.	0.8	2	3	11	76	0.8	—
Dallas	Commerce St.	0.6	5	10	8 ·	68	0.9	—
Chicago	63rd St.	10.3	2	2	93	40	1.5	904
Atlanta	Peachtree St.	0.3	2	6	3	66	0.9	770
New York ·	Fulton St.	0.6	2	5	5	75	0.8	—
St. Louis	Washington St. ·	1.5	3	4	13	30	2.0	572
New Orleans	Baronne St.	0.7	2	3	6	45	1.4	722
New Orleans	Tulane Ave.	0.7	3	1	7	30	2.0	1,398

[a] Prevailing direction only.

TABLE 11.3—OBSERVED PEAK-HOUR VOLUMES OF LOCAL BUSES ON CITY STREETS WITH RESERVED TRANSIT LANES[a]

CITY ·	FACILITY	LENGTH OF SEC-TION (MI)	TRAFFIC LANES, ONE DIREC-TION	TRANSIT ROUTES USING STREET	SERVICE STOPS IN SEC-TION	BUS MOVEMENT		AUTO-MO-BILES
						NO. OF BUSES	HEAD-WAY (MIN)	
Rochester	Main St.	0.5	3	9	7	93	0.6	932
Chicago	Washington Blvd.	0.5	5·	5	7	66	0.9	1,152
Atlanta	Peachtree St.	0.3	3	6	3	67	0.9	1,100
Dallas	Commerce St.	0.6	5	10·	8	67	0.9	—
Birmingham	2nd Ave., North	0.8	4	8	7	44	1.4	1,413
Baltimore	Charles St.	2.1	3	2	22	38	1.6	—

[a] Prevailing direction only.

Thus, buses benefit from highway facilities providing free and swift travel, but are at the same time subject to highway delays.

Buses moving on highways and streets in mixed traffic usually constitute only a small percentage of the vehicular traffic. Examples of peak-hour volumes of buses under various conditions are provided in Tables 11.2 through 11.6. The observations reported in these tables do not represent maximum possible bus volumes or maximum total traffic volumes. Highway capacity under mixed

TABLE 11.4—OBSERVED PEAK-HOUR VOLUMES OF EXPRESS BUS SERVICE ON CITY STREETS[a]

CITY	FACILITY	LENGTH OF SEC-TION (MI)	TRAFFIC LANES, ONE DIREC-TION	TRANSIT ROUTES USING STREET	SERVICE STOPS IN SEC-TION	BUS MOVEMENT		AUTO-MO-BILES
						NO. OF BUSES	HEAD-WAY (MIN)	
St. Louis	Gravois St.	1.3	5	7	2	66	0.9	1,531
Cleveland	Clifton Blvd.	5.0	3	1	22	32	1.9	1,803
Chicago	Archer Ave.	11.0	4	1	33	29	2.1	700
San Francisco	Van Ness/Broadway/ Stockton	1.9	1–3	1	1	17	3.5	1,540
New Orleans	Earhart Blvd.	2.0	2	2	0	25	2.4	1,357

[a] Prevailing direction only.

TABLE 11.5—OBSERVED PEAK-HOUR VOLUMES OF EXPRESS BUS SERVICE ON EXPRESSWAYS[a]

CITY	FACILITY	LENGTH OF SEC-TION (MI)	TRAFFIC LANES, ONE DIREC-TION	TRANSIT ROUTES USING STREET	SERVICE STOPS IN SEC-TION	BUS MOVEMENT		AUTO-MO-BILES
						NO. OF BUSES	HEAD-WAY (MIN)	
Chicago	Lake Shore Dr.	3.0	6	8	0	99	0.6	3,463
Cleveland	Shoreway West	3.2	4	1	1	32	1.9	6,340
San Francisco	Bayshore Fwy.	2.8	3–4	3	0	35	1.7	6,800
Los Angeles	Hollywood Fwy.	4.0	4	5	1	41	1.5	8,010
St. Louis	Mark Twain Expwy.	11.8	2–4	8	4	52	1.2	4,639
Atlanta	North Expwy.	1.5	3	6	0	19	3.2	4,915
Dallas	Central Expwy.	1.2	3	6	0	30	2.0	4,380
St. Louis	3rd St. Expwy.	1.6	3	4	0	29	2.1	3,600
Philadelphia	Schuylkill Expwy.	6.0	3	1	0	18	3.3	4,335
St. Louis	Daniel Boone Expwy.	6.5	2	2	3	10	6.0	3,905

[a] Prevailing direction only.

TABLE 11.6—OBSERVED PEAK-HOUR VOLUMES OF EXPRESS BUS SERVICE ON TERMINAL RAMPS, TUNNEL APPROACHES, TUNNELS, AND BRIDGES[a]

CITY	FACILITY	LENGTH OF SECTION (MI)	TRAFFIC LANES, ONE DIRECTION	TRANSIT ROUTES USING STREET	SERVICE STOPS IN SECTION	BUS MOVEMENT		AUTOMOBILES
						NO. OF BUSES	HEADWAY (MIN)	
New York	P.A. Bus Terminal	0.3	2	53	0	511	0.12	—
Union City, N.J.	Route 3	0.4	3	50	0	397	0.15	2,753
New York	Lincoln Tunnel	1.5	2	50	0	527	0.11	1,882
San Francisco	S.F.-Oakland Bay Br.	5.2	5	14	0	216	0.28	6,185
New York	Geo. Washington Br.	1.1	4–5	28	0	136	0.44	3,659

[a] Prevailing peak flow direction only.

traffic conditions is treated in the previous sections of this manual by adjustment of passenger car capacity based on the per-

Extensive use of curb lanes by transit in a central business district with heavy pedestrian traffic.

centage of through or local buses in the traffic stream. For uninterrupted flow conditions this is covered in Chapter Five; for intersection capacity, in Chapter Six. Regarding the intersection procedures, it should be remembered that the bus adjustment procedures contained therein apply only to local transit buses making stops at the curb, through express buses being considered as trucks.

At some locations on the highway system bus movements are concentrated and exceed percentage ranges of commercial travel discussed in other sections of this manual. For example, heavy movements of through buses occur at river crossings in or near major cities, and bus movements are concentrated on principal downtown area streets.

Uninterrupted Flow

Studies of concentrated bus movements have have been made to determine bus-car flow capacity relationships. For instance a study was made in June 1962 by the Port of New York Authority in a single lane of the 2-lane, one-way, north tube of the Lincoln Tunnel (3). The study site was on a level section at approximately the midpoint of the $1\frac{1}{2}$-mile long tunnel. Automatic detecting and recording equipment was used to determine the time of passage of the front and rear of each vehicle over two points a few feet apart. A computer program then sum-

Predominantly transit street. Absence of automobile traffic facilitates bus pullout after loading passengers.

marized a variety of vehicle characteristics, including velocity, length, and headway time.

In this single lane carrying 60 percent cars, 32 percent buses and 8 percent trucks, data were collected on 3,200 vehicles. Included were 1,200 samples where cars followed cars, and almost 400 cases of buses followed by buses. The relationship of time headway and speed were compared for these two types of flow. Results showed for cars following cars a minimum headway of 2.39 sec at 21.6 mph; for buses following buses, a minimum of 3.49 sec at 24.2 mph. The headway difference was found to range from 1.3 sec at speeds of 14 mph to 1.0 sec at 41 mph, with a 1.1-sec difference of minimum values.

Comparison of the minimum headway times resulted in a bus equivalent of 1.46 cars. Over a speed range of 14 to 41 mph the equivalent was found to decrease from 1.53 to 1.36, probably because the greater length of buses is a more significant influence at low speeds. In summary, it was found that a car-bus equivalent varies with speed but that an equivalent of 1.5 cars per bus is representative of tunnel flow.

A recent broad nationwide study by the Bureau of Public Roads of mixed traffic

flows on expressways carrying relatively large numbers of buses (4), involving detailed recording of speed and spacing of many thousands of vehicles, has indicated a factor of 1.6 as generally applicable on both expressways and full freeways. This factor appears equally appropriate to each of the traffic lanes at their normal operating speeds.

Locations observed in this study included:

1. Route 3 approaches to Lincoln Tunnel, New Jersey (New York City area).
2. Center Tube, Lincoln Tunnel, New Jersey (New York City area).
3. Shoreway West, Cleveland, Ohio.
4. Lakeshore Drive, Chicago, Ill.
5. Mark Twain Expressway, St. Louis, Mo.
6. Bayshore Freeway, San Francisco, Calif.
7. San Francisco-Oakland, California, Bay Bridge (temporary exclusive bus lane, lower deck).

An earlier limited study of mixed traffic conducted by the Bureau of Public Roads on the Shirley Highway near Washington, D.C., showed a 1.7 factor, and a recent test track study by a bus manufacturer showed a value of about 1.4.

Bus lane (curb lane) reserved for transit operations.

The similarity of these several findings indicates that when buses are in motion, either in exclusively bus traffic or in mixed traffic, under uninterrupted flow conditions over a broad range of levels of service, their equivalency factor will be approximately 1.6 passenger cars.

Uninterrupted Exclusive Lane Flow

The capacity or service volume of an exclusive bus lane with uninterrupted flow can be computed by applying the 1.6-car equivalency factor to the computed capacity or corresponding service volume in passenger cars per hour. For example, a roadway lane having a capacity of 1,500 passenger cars per hour would have an equivalency of 940 buses per hour and one within level of service C at a service volume of 1,100 cars per hour, an equivalency of 690 buses per hour. This uninterrupted flow volume requires, of course, in the case of a single-lane facility, that bus stops be located off the lane and that adequate acceleration and deceleration lanes be provided.

Interrupted Flow (Intersection Capacity)

The effect of transit buses on intersection approach volumes is dependent on several factors, such as their number, whether or not they make local stops, and the location of the stops. These effects are discussed in Chapter Six. The previously mentioned equivalency factor of 1.6 is not applicable to interrupted flow conditions, and evidence thus far developed indicates that no single all-inclusive factor can be developed.

Reserved Transit Lanes on City Streets

A number of cities have established, or are considering, reserved transit lanes to improve the flow of highway traffic. Because buses stop within these lanes to pick up and discharge passengers, the ability of these lanes to move traffic probably will be affected by loading and unloading time requirements presented in a following section. At present, few research data have been collected concerning the improved operation of reserved transit lanes. Meaningful "before" and "after" comparisons have not been possible, in many cases, because establishment of the
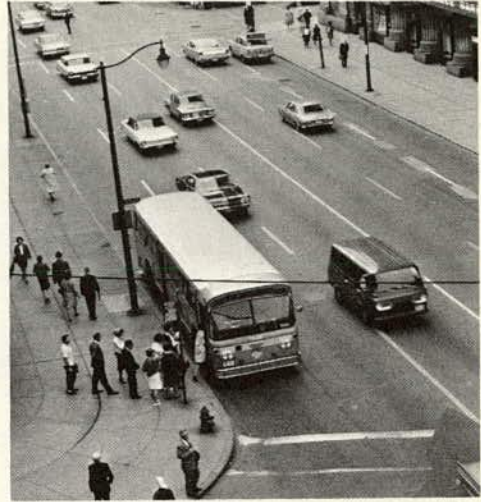
Bus loading of passengers at near-side bus stop.
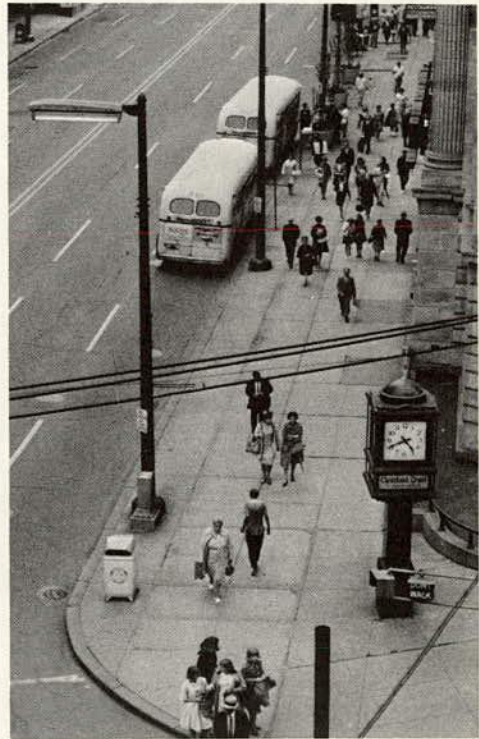
Far-side bus stop providing loading zone for several buses.

all-bus lane was only part of an overall up-
grading of the street which included new
traffic engineering devices and sometimes
conversion from two-way to one-way opera-
tion. In at least one case a two-way arterial
was converted to favor bus transit operation,
through timing of the progressive signal sys-
tem to take into consideration average time
delays at bus stops, while parallel streets on
either side were signalized to favor passenger
car operation. In any reserved lane situa-
tion, a highly significant consideration is
whether or not buses are rigidly restricted to
the exclusive lane. If they are, undesirable
bus queuing may develop, delaying bus op-
erations; if they are not, passing movements
may adversely influence other traffic lanes.
To aid officials considering reserved transit
lanes, the Institute of Traffic Engineers has
drafted suggested warrants for their adop-
tion (5).

Data concerning bus and automobile
volumes on streets with presently reserved
lanes are given in Table 11.3; these do not
necessarily represent maximum possible
volumes.

TABLE 11.7—PASSENGER INTERVAL
(SERVICE TIME) ON AND OFF BUSES

OPERATION	CONDITIONS	TIME[a] (SEC)
Unloading	Very little hand bag-gage and parcels; few transfers	1½-2½
	Moderate amount hand baggage or many transfers	2½-4
	Considerable baggage from racks (intercity runs)	4-6
Loading	Single coin or token fare box	2-3
	Odd-penny cash fares	3-4
	Multiple-zone fares; prepurchased tickets and registration on bus	4-6
	Multiple-zone fares; cash, including regis-tration on bus	6-8

[a] Per door.

PERFORMANCE AT PICK-UP/DISCHARGE POINTS

Bus Stops on City Streets

As discussed in Chapter Six, the location
of curb bus stops—near side of intersection,
far side, or midblock—may have a signifi-
cant effect on the transit operation itself, as
well as on overall street capacity. No rigid
criteria can be established. An efficient
transit operation which is in harmony with
the overall traffic flow requires individual
analysis of each stop installation.

Where bus arrivals at an intersection are
such that they normally encounter a red
signal, a near-side stop is desirable to com-
bine signal delay with loading/unloading
delay; a faster overall transit operation thus
results. On the other hand, where a green
signal usually is encountered on arrival,
which would turn red by the time loading/
unloading was completed, a far-side stop will
permit the fastest operation. These are not
the only criteria, however. For instance, the
needs of right-turning vehicles must also be
considered, and the obstructions resulting
from buses in a near-side stop must be
weighed against the additional value of the
bus stop as a right-turn lane when not oc-
cupied by a bus. The decision here usually
will depend on the frequency with which
buses use the particular stop. Convenience
to transferring passengers at intersections of
bus routes is still another consideration.

The service volume of a bus route may be
limited by the ability of the stops to handle
the picking up and discharging of passengers.
Similarly, the capacity itself may be thus
limited, if inadequate space for stops exists.
Each vehicle requires a certain amount of
"service time" at the stop, varying with the
number of boarding and alighting passen-
gers. The average headway between vehicles
using each loading position at a stop to
handle passengers, therefore, depends on the
number of boarding and alighting passengers
and on the number of loading positions. In
addition, volume is somewhat increased if
vehicles can overtake each other when enter-
ing or leaving loading positions.

A detailed analysis of service times is be-
yond the scope of this discussion. Table 11.7

Exclusive bus loading area adjacent to a freeway. Note fencing for passenger protection.

summarizes loading and unloading time per passenger per door, from which average service time per vehicle may be calculated.

The number of buses that can be handled at curbside bus stops without unacceptably long queues (and associated waiting lines) being caused varies principally with this service time per bus and, to a lesser degree, with the number of loading positions. Additional loading spaces (or additional length of bus zones) increases the capacity, but at a decreasing rate as the number of spaces increases. No full statistical analysis of these relationships has yet been achieved, but data from actual operations indicate that a bus stop can serve buses arriving at half the average service rate, or trip frequency, with well under 10 percent probability of forming queues beyond the stop.

Until additional knowledge of the underlying statistical phenomena is available, an acceptable rule of thumb might be to assume that the headways at a curbside bus stop (in minimum seconds of interval between vehicles) could be about twice the average service time per vehicle. Along any artery, the stop with the longest service time will be the bottleneck. The capacity of the artery itself could be increased by providing different bus stops for different routes, provided vehicles could overtake each other. To illustrate: Assume that along "Main Street" the average service time at the busiest bus stop is 25 sec. Provided the length of the bus stop is adequate, this stop will handle buses at a minimum headway of about 50 sec. Headways can be approximately halved (frequency of service doubled) by providing alternate sets of bus stops far enough removed from each other so as not to cause interference in entering and leaving the loading zones. Each set of stops can then handle

TABLE 11.8—MINIMUM DESIRABLE LENGTHS FOR BUS CURB LOADING ZONES [a]

APPROX. BUS SEATING CAPACITY	APPROX. BUS LENGTH (FT)	LOADING ZONE LENGTH [b] (FT)					
		ONE-BUS STOP			TWO-BUS STOP		
		NEAR SIDE [c]	FAR SIDE [d]	MID-BLOCK	NEAR SIDE [c]	FAR SIDE [d]	MID-BLOCK
30 and less	25	90	65	125	120	90	150
35	30	95	70	130	130	100	160
40-45	35	100	75	135	140	110	170
51-53	40	105	80	140	150	120	180

[a] Source: American Transit Association.
[b] Measured from extension of building line, or from an established stop line, whichever is appropriate. Based on side of bus positioned 1 ft from curb; if bus is as close as 6 in. from curb, 20 ft should be added to near-side stops, 15 ft to far-side stops, and 35 ft to midblock stops.
[c] Increase 15 ft where buses are required to make a right turn. If there is a heavy right-turn movement of other vehicles, near-side stop zone lengths should be increased 30 ft.
[d] Based on roadways 40 ft wide, which enable buses to leave the loading zone without passing over centerline of street. Increase 15 ft if roadway is 36 ft wide, and 30 ft if roadway is 32 ft wide.

buses at 50-sec headways, and the street as a whole can handle buses at 25-sec headways, if exactly 50 percent of the buses are assigned to each set of stops, and if schedule reliability can be maintained. Of course, ample smoothly-operating stops help assure schedule reliability. However, it should be realized that in the case of the usual all-bus-lane operation, buses would be restricted to this lane, hence overtaking would be impossible and multiple stops would not be feasible.

Table 11.8 gives the minimum desirable lengths for bus curb-loading zones, for one- and two-bus loading conditions.

Bus Stops on Freeways

Bus loading zones on an exclusive roadway within a freeway right-of-way have capacities similar to those of curbside loading zones. Here again, the length of the stop and the ability of buses to overtake others are important. Given similar loading facilities, any difference does not lie in the operation of the stop itself, but in the capacity of the roadway lane leading into and away from the stop.

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APPENDIX A

VARIATIONS IN TRAFFIC FLOW ON ACTUAL HIGHWAYS IN THE UNITED STATES

TABLE A.1—VARIATIONS IN TRAFFIC FLOW ON RURAL FREEWAYS

LOCATION OF COUNT STATION	NO. OF LANES	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
				AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
New England													
Maine													
1.9 mi from Yarmouth	4	I 95	EB			16.1	14.0	13.4	13.0	12.6	12.2	—	—
			WB			17.4	14.6	13.4	12.9	12.5	12.2	—	—
			Both	9,457	13,402								
1.3 mi from Augusta	4	I 95	NB			20.4	14.1	13.4	12.9	12.6	12.2	—	—
			SB			21.6	17.4	16.2	14.9	14.4	14.0	—	—
			Both	4,247	8,118[c]								
New Hampshire													
0.5 mi from Manchester	4	US 3	Both	10,424	20,643[c]	23.4	17.1	15.7	14.9	14.1	13.8	—	—
3.0 mi from Concord	4	I 93	Both	7,536	17,863[c]	22.5	21.0	19.9	19.0	17.8	16.9	14.8	—
Rhode Island													
1.0 mi from Warwick	4	I 95	NB			13.3	11.7	9.8	8.7	—	—	—	—
			SB			11.2	9.6	—	—	—	—	—	—
			Both	4,933	6,179	10.8	9.4	8.6	8.2	7.9	7.4	—	—
Vermont													
5 mi from Brattleboro	4	I 91	NB			33.6	26.2	25.3	23.9	22.3	20.0	15.4	—
			SB			40.2	33.4	29.7	26.5	25.2	24.0	20.0	—
			Both	3,971	9,207[c]	25.2	20.7	20.0	18.7	17.7	17.1	15.8	—
4 mi from Montpelier	4	I 89	NB			21.3	15.6	14.1	13.3	13.0	12.6	11.0	—
			SB			16.6	13.9	12.3	12.0	11.9	11.7	11.0	—
			Both	3,962	5,463	16.2	13.0	12.4	12.1	11.7	11.4	10.7	—
10 mi from Bellows Falls	4	91	NB			25.5	19.8	18.2	17.1	16.0	15.5	13.8	—
			SB			37.7	31.2	29.0	26.0	24.1	23.3	19.9	—
			Both	2,466	5,711[c]								
Middle Atlantic													
New York													
0.2 mi W of NYS 208	4	NYS 17	Both	12,500[b]	—	23.4	19.6	16.6	14.9	12.8	10.9	7.9	5.5
Pennsylvania													
2.5 mi N. of Allentown	4	I 78	EB			10.9	9.4	8.9	8.8	8.7	8.6	—	—
			WB			14.1	12.4	12.0	11.6	11.4	11.2	—	—
			Both	30,594	40,019	11.4	10.1	9.8	9.6	9.4	9.3	—	—

Location	Lanes	Route	Dir	Vol 1	Vol 2								
2.5 mi N of York	4	I 83	NB			15.2	12.7	11.9	11.5	11.2	11.1	—	—
			SB			16.6	14.2	13.6	13.3	12.8	12.1	—	—
			Both	11,855	17,685	14.3	12.2	11.7	11.5	11.2	10.9	—	—
South Atlantic													
Maryland													
Baltimore-Washington Expwy. S of Md. 176, 2.2 mi from Dorsey	4		NB			13.8	12.8	12.6	12.4	—	11.8	—	—
			SB			12.4	11.2	10.8	10.5	—	10.3	—	—
			Both	28,889	—	11.6	10.0	9.7	9.6	—	9.7	—	—
Virginia													
0.3 mi S of Rt 659 near Staunton		Rt 81	NB			24.2	17.7	15.1	13.6	13.2	13.0	11.8	10.5
			SB			17.2	15.1	13.7	12.3	12.0	11.4	10.1	8.9
			Both	4,179	9,108c	16.1	13.6	13.1	13.0	11.3	11.1	10.1	9.2
North Carolina													
14 mi from Greensboro	4	I 85	Both	9,560	15,400c	15.9	13.0	12.2	11.8	11.3	11.1	10.1	—
4 mi from Salisbury	4	I 85	Both	9,980	13,869c	19.6	11.6	10.9	10.3	9.8	9.4	8.4	—
9 mi from Greensboro	4	US 29	Both	6,740	10,515c	18.0	11.2	10.5	10.4	10.2	10.1	9.3	—
10 mi from Fayetteville	4	I 95	Both	5,680	12,004c	16.2	12.8	11.8	11.3	10.9	10.7	10.0	—
South Carolina													
Pacolet R. near Spartanburg	4	I 85	Both	6,319	10,096c	16.1	11.4	10.5	9.8	9.7	9.5	8.9	8.1
East North Central													
Illinois													
4.2 mi from South Holland	4	FAP 122	NB			18.0	15.4	14.7	14.6	14.1	13.9	13.0	12.3
			SB			15.6	14.6	14.0	13.1	12.9	12.7	12.3	11.6
			Both	14,000	20,890c	12.0	11.4	10.9	10.6	10.4	10.3	10.1	10.0
Michigan													
1.0 mi from New Hudson	4	I 96	EB			34.4	30.3	28.7	26.6	24.7	23.5	19.1	14.3
			WB			34.9	29.5	26.0	23.7	22.5	20.9	16.7	13.7
			Both	17,329	47,212c	22.0	20.2	18.9	18.4	17.8	17.1	14.9	13.0
1.0 mi from Romulus	4	I 94	EB			16.6	14.4	13.6	12.7	12.3	11.9	10.8	9.8
			WB			17.1	14.0	11.5	11.1	10.8	10.5	9.8	9.3
			Both	27,437	37,291	14.8	11.8	10.9	10.7	10.5	10.3	9.7	9.1
3.5 mi from Monroe	4	I 75	NB			14.9	12.6	11.8	10.9	10.6	10.3	9.4	8.4
			SB			15.0	12.9	12.1	11.8	11.2	11.0	10.3	9.3
			Both	14,240	27,368c	14.4	12.1	11.3	10.9	10.6	10.4	9.6	8.7
8 mi from The Heights	4	US 27	NB			43.1	37.1	34.2	32.6	31.6	29.9	26.0	20.5
			SB			76.0	52.2	47.5	45.0	43.3	41.1	32.2	24.4
			Both	4,727	18,748c	40.7	34.7	32.4	30.8	29.9	29.0	25.8	19.9
2.0 mi from New Buffalo	4	I 94	NB			22.6	20.0	18.7	17.6	17.0	16.3	14.5	12.7
			SB			51.4	29.8	26.3	23.8	22.4	21.1	15.7	13.4
			Both	8,018	22,888c	28.1	21.1	19.3	18.8	17.3	16.9	14.9	12.7

LOCATION OF COUNT STATION	NO. OF LANES	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
				AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
Michigan (Cont'd)													
4.0 mi E of Marshall	4	I 94	EB			18.8	16.0	14.7	13.9	13.4	13.1	11.7	10.0
			WB			16.7	14.0	12.9	12.4	12.0	11.7	10.9	10.0
			Both	9,435	16,467c	17.4	14.2	13.2	12.6	12.2	11.9	10.9	9.8
3 mi from Alto	4	I 96	EB			35.3	17.8	16.2	15.5	15.0	14.5	12.6	10.6
			WB			31.2	15.2	13.7	12.7	12.0	11.6	10.8	9.8
			Both	6,061	9,302c	21.1	15.2	13.5	13.1	12.6	12.3	11.2	10.0
5.6 mi from St. Ignace	4	I 75	NB			29.8	26.2	25.1	24.1	23.5	23.2	21.1	18.6
			SB			24.8	22.1	21.2	20.4	20.1	19.6	18.2	16.5
			Both	2,736	7,645c	24.8	22.3	21.6	21.2	20.4	20.1	18.8	17.6
Ohio													
5.1 mi from Ashland	4	I 71	NB			22.4	18.6	16.6	15.5	15.2	14.8	—	—
			SB			22.9	16.7	14.6	13.8	13.0	12.5	—	—
			Both	10,019	21,005c	21.0	16.4	14.5	13.9	13.4	13.0	11.8	10.6
3.1 mi from North Kingsville	4	I 90	EB			18.6	15.5	14.8	14.4	14.2	14.0	13.2	12.0
			WB			26.2	20.7	19.3	17.7	17.0	16.5	14.8	13.2
			Both	6,624	15,277c	17.8	16.4	15.6	15.3	14.9	14.7	13.8	12.5
2.0 mi from Wapakoneta	4	I 75	NB			16.9	12.7	11.7	11.2	10.8	10.5	9.9	9.0
			SB			15.9	13.0	12.4	12.0	11.7	11.3	10.3	9.5
			Both	7,685	13,441	15.5	12.3	11.5	11.3	11.1	10.8	9.8	9.0
Wisconsin													
11.1 mi from Milwaukee	4	I 94	EB			17.3	14.7	13.9	13.2	12.3	11.9	11.0	10.2
			WB			21.3	16.7	15.5	14.5	14.0	13.8	12.2	11.2
			Both	17,860	32,820c	15.2	13.7	13.1	12.4	12.2	11.9	11.2	10.3
27 mi from Madison	4	I 90-94	NB			28.2	22.6	19.7	19.2	18.5	18.2	15.8	13.4
			SB			44.7	32.1	27.1	25.5	24.4	23.4	19.8	16.1
			Both	6,317	16,384c	26.4	20.2	18.9	18.0	17.2	16.8	15.6	14.0
West North Central													
Iowa													
S of Fuller Rd. in West Des Moines	4	I 35	NB			19.4	16.2	15.4	14.9	14.4	14.0	—	—
			SB			16.5	14.3	13.3	13.0	12.7	12.3	—	—

Location													
3 mi N of Crescent Interchange	4	I 29	Both	5,517	9,575c	17.7	14.6	13.9	13.5	13.1	12.9	—	—
			NB			18.8	16.3	14.8	13.6	12.9	12.3	—	—
			SB			23.5	20.6	19.6	18.9	18.2	17.2	—	—
			Both	4,646	8,815c	15.7	14.6	13.8	13.4	13.2	12.9	—	—
Kansas													
5.5 mi from Maple Hill	4	I 70	Both	4,465	7,811c	17.7	15.3	14.2	13.5	13.1	12.8	11.6	—
Missouri													
20 mi from St. Louis	4	I 44	WB	14,835	31,911c	25.2	17.3	15.1	14.6	14.1	13.8	10.8	9.8
15 mi NE of Springfield	4	I 44	WB	6,983	10,956c	13.6	11.9	11.4	11.0	10.7	10.7	10.0	9.3
Nebraska													
11 mi SW of Omaha	4	I 80	NB			30.4	18.9	16.1	15.1	13.3	13.2	11.8	10.0
			SB			29.9	19.4	15.7	13.3	12.6	12.2	10.3	9.6
			Both	7,030	12,296c	18.2	14.6	13.1	12.5	11.9	11.6	10.6	9.5
North Dakota													
2 mi from Mapleton	4	I 94	EB			18.3	14.8	13.1	12.2	11.9	11.6	—	—
			WB			19.7	13.9	13.1	12.5	12.1	11.8	—	—
			Both	3,758	6,000	15.1	12.3	11.6	11.1	10.8	10.4	—	—
1 mi from SW Fargo	4	I 94	EB			25.5	18.2	17.0	16.4	15.5	15.4	—	—
			WB			22.4	16.9	15.2	13.5	12.2	11.7	—	—
			Both	2,486	5,244	18.5	15.5	15.0	14.7	14.2	9.9	—	—
3 mi from Buffalo	4	I 94	EB			26.5	14.5	13.6	13.0	12.6	12.2	—	—
			WB			25.3	15.1	13.7	13.2	12.9	12.7	—	—
			Both	2,626	4,367c	16.4	13.1	11.9	11.5	11.3	11.1	—	—
4.5 mi from Sanborn	4	I 94	EB			24.7	17.2	14.5	13.3	12.9	12.6	—	—
			WB			21.0	16.0	14.9	14.3	13.5	13.1	—	—
			Both	2,540	4,469c	16.6	14.8	13.9	12.9	12.4	12.2	—	—
2 mi from Medina	4	I 94	EB			24.5	15.1	14.1	13.5	13.0	12.8	—	—
			WB			35.1	18.7	16.9	15.9	15.4	14.7	—	—
			Both	1,741	3,641c	22.2	14.7	13.9	13.4	13.1	12.7	—	—
West South Central													
Arkansas													
5 mi from Benton	4	I 30	EB			16.7	13.0	12.0	11.4	10.9	10.5	9.3	8.1
			WB			19.0	15.8	11.6	11.0	10.6	10.4	9.7	8.7
			Both	8,850	11,375c								
Oklahoma													
N of Oklahoma City	4	I 35	Both	7,650	—	16.7	12.3	11.8	11.4	11.1	10.7	9.8	7.3
7 mi from Canute	4	I 40	Both	4,574	—	14.0	10.8	10.5	9.0	8.4	8.1	7.0	—
Texas													
S of Austin, Travis Cty.	6	I 35	NB			20.9	14.4	12.8	12.0	11.7	11.4	10.4	9.2
			SB			35.3	13.7	12.2	11.7	11.4	11.2	10.4	9.5
			Both	8,000	11,957c								

LOCATION OF COUNT STATION	NO. OF LANES	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
				AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
Texas (Cont'd)													
5 mi from Corsicana	4	I 45	NB			20.3	15.7	14.1	13.1	12.5	12.3	11.2	9.9
			SB			21.8	14.0	12.9	12.1	11.6	11.2	10.2	9.0
			Both	5,460	8,974[c]								
Mountain													
Arizona													
5 mi from Bensor	4	I 10	EB			13.0	8.6	8.2	8.2	8.1	8.1	8.1	8.0
			WB			15.6	10.1	9.6	9.5	9.3	9.2	9.2	9.1
			Both	4,397	8,188[c]								
4 mi from Gila Bend	4	I 10	EB			15.1	9.5	8.7	8.7	8.5	8.3	8.2	8.1
			WB			17.6	9.2	8.7	8.6	8.5	8.4	8.4	8.3
			Both	3,537	6,775[c]								
Colorado													
20 mi from Denver	4	I 25	NB			26.4	18.0	16.2	15.0	14.4	13.8	12.0	10.5
			SB			23.9	18.6	17.6	16.7	16.0	15.3	13.2	10.6
			Both	8,223	14,609[c]	16.5	14.0	13.3	12.9	12.5	12.1	11.2	10.0
8 mi N of Pueblo	4	I 25	NB			19.4	17.4	15.5	14.2	13.7	13.2	12.2	10.9
			SB			16.2	14.2	13.6	13.1	12.6	12.1	11.3	10.3
			Both	6,810	12,357[c]	16.0	14.2	13.7	13.0	12.7	12.3	11.2	10.2
Idaho													
24.6 mi from American Falls	4	I 15W	NB			18.4	14.5	14.0	13.4	12.6	12.0	11.2	10.1
			SB			24.0	22.4	21.6	20.7	20.1	19.8	17.8	13.8
			Both	1,678	3,165[c]	15.7	13.8	13.4	12.9	12.6	12.5	11.7	11.0
Montana													
4.0 mi W of Butte	4	I 90	NB			20.3	16.3	15.6	14.6	13.9	13.6	12.5	—
			SB			22.2	15.1	13.8	13.0	12.4	12.0	10.6	—
			Both	4,906	8,814[c]	13.9	12.5	12.0	11.8	11.7	11.5	10.9	—
7.0 mi W of Billings	4	I 90	EB			21.8	17.3	16.7	15.8	15.3	14.8	10.5	—
			WB			16.6	11.6	10.8	10.2	9.8	9.7	8.7	—
			Both	4,289	6,793[c]	12.6	11.7	11.1	10.8	10.4	10.3	9.6	—
8.0 mi W of Great Falls	4	I 15	NB			22.7	17.1	15.4	14.7	14.0	13.4	11.1	—

Location		Route	Dir.										
			SB			37.0	13.5	12.7	12.1	11.6	11.0	10.2	—
			Both	4,092	6,345c	19.6	12.0	11.6	11.1	10.8	10.7	10.1	—
Nevada													
30 mi from Las Vegas	4	I 15	NB			18.8	16.3	15.6	14.8	13.9	13.5	11.9	10.4
			SB			32.7	19.9	18.4	17.4	16.4	16.1	14.3	12.4
			Both	6,409	12,237c	19.1	13.3	12.3	11.9	11.5	11.2	10.3	9.3
Wyoming													
4.0 mi from Rawlins	4	I 80	Both	2,999	5,987c	13.5	12.9	12.4	12.1	11.9	11.7	—	—
Pacific													
Oregon													
Pacific Hwy. 15, 5 mi N of Salem	4		NB			17.6	14.7	13.4	12.8	12.6	12.3	11.5	—
			SB			26.6	14.0	12.9	12.3	11.9	11.5	10.8	—
			Both	12,871	21,487c	16.8	13.0	12.3	11.9	11.5	11.3	10.6	—
1 mi from Troutdale	4	I 80N	EB			32.0	24.2	21.4	19.2	18.0	17.1	—	—
			WB			30.7	28.3	24.9	23.5	22.3	21.7	17.6	—
			Both	7,096	18,431c	23.4	21.5	20.0	18.7	17.7	17.2	—	—
Washington													
4.8 mi W of Olympia; Olympia leg of junction	4	US 101 US 410	EB			34.0	28.5	26.1	24.5	23.5	21.9	17.2	13.6
			WB			17.6	16.2	15.5	15.1	14.8	14.4	13.2	11.2
			Both	8,125	18,974c	20.7	17.9	16.8	16.4	15.6	15.2	13.3	11.9
5.2 mi from Centralia	4	I 5	NB			23.3	17.0	14.9	14.3	13.9	13.8	12.4	11.1
			SB			16.8	15.7	14.7	13.9	13.5	13.3	12.6	11.6
			Both	10,728	20,542c	15.1	13.6	13.2	12.8	12.6	12.4	11.7	10.9
11.4 mi N of Everett	4	I 5	NB			16.5	14.2	13.6	13.1	12.8	12.6	11.5	10.0
			SB			21.1	19.1	18.4	17.3	16.9	16.7	14.5	12.0
			Both	10,915	20,469c	14.4	13.4	13.0	12.8	12.6	12.3	11.6	10.5
4.8 mi from Olympia	4	US 101 US 410	EB			45.5	39.7	35.5	32.7	31.0	29.3	22.4	17.1
			WB			28.8	21.3	20.5	19.5	18.6	18.0	16.1	13.2
			Both	3,769	11,292	26.9	23.7	22.1	20.8	19.7	18.4	16.4	14.1
4.8 mi W of Olympia	4	US 101 US 410	NB			22.0	14.7	14.0	13.6	13.3	13.0	12.1	11.1
			SB			31.9	19.8	18.5	17.7	16.9	16.3	13.7	11.0
			Both	4,356	7,682c	19.3	13.9	13.4	13.1	12.6	12.3	11.5	10.5

a For calendar year 1962 except as noted. b For calendar year 1961. c Peak day occurred on a Saturday or Sunday.

TABLE A.2—VARIATIONS IN TRAFFIC FLOW ON RURAL EXPRESSWAYS

LOCATION OF COUNT STATION	NO. OF LANES	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
				AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
New England													
Rhode Island													
Sakonnet River Bridge	4	RI 138	NB			18.6	16.9	13.4	10.8	—	—	—	—
			SB			17.3	13.3	11.5	10.7	—	—	—	—
			Both	11,812	20,852[c]	15.6	13.7	12.5	11.4	10.8	10.5	—	—
Middle Atlantic													
New Jersey													
6 mi from Dover	4	US 46	EB			19.2	18.7	18.6	18.0	17.5	16.9	14.5	12.3
			Both	25,438[b]	—								
1 mi from Ramsey	4	Rt 17	SB			20.9	20.3	20.2	19.7	19.4	17.9	16.1	11.7
			Both	21,414[b]	—								
2.5 mi from Pompton Lakes	4	Rt 23	EB			20.1	18.9	18.7	17.4	16.2	16.1	15.2	13.6
			Both	22,066[b]	—								
New York													
21 mi N of Utica	4	NYS 12 NYS 28	Both	4,900[b]	—	50.8	27.1	22.9	21.4	20.2	19.4	15.9	12.2
10 mi from Saratoga	4.	US 9	Both	9,700[b]	—	18.4	15.6	13.4	12.3	11.4	10.9	8.1	6.2
South Atlantic													
Maryland													
1.3 mi from Aberdeen	4	US 40	NB			11.2	10.0	9.7	9.4	—	8.9	—	—
			SB			11.4	10.2	9.8	9.7	—	9.3	—	—
			Both	28,693	—	10.7	9.5	9.1	8.8	—	8.5	—	—
1.2 mi S of Waldorf	4	US 301	NB			15.3	13.6	13.1	12.4	—	11.8	—	—
			SB			20.7	16.1	13.4	12.5	—	11.5	—	—
			Both	12,437	—	14.9	11.9	10.8	10.5	—	9.8	—	—
West Virginia													
4.8 mi from Charleston	4	US 60	EB			13.4	12.0	11.6	11.4	11.1	10.8	—	—
			WB			19.2	13.8	11.9	11.2	10.9	10.6	—	—
			Both	16,125	26,079								

East North Central
Michigan

Location		Route	Dir										
8 mi from Perry	4	M 78	NB			27.5	14.3	12.5	11.8	11.7	11.4	10.7	9.8
			SB			26.3	15.4	12.8	11.6	11.3	11.0	10.0	9.2
			Both	7,271	9,985	19.1	12.5	11.6	11.2	10.7	10.5	9.9	9.1
2 mi S of Mason	4	US 127	NB			23.1	13.2	12.4	11.7	11.5	11.2	10.5	9.3
			SB			22.6	15.4	14.2	13.8	13.4	12.9	11.9	10.7
			Both	6,815	10,661ᶜ	16.1	13.2	12.5	12.1	11.8	11.4	10.8	9.7

Ohio

Location		Route	Dir										
4.6 mi from Vienna	4	US 40	EB			17.8	14.5	12.4	12.2	12.0	11.6	10.8	10.1
			WB			17.8	13.8	13.1	12.2	11.8	11.5	10.6	9.7
			Both	10,220	18,092ᶜ	15.6	12.7	12.1	11.6	11.4	11.2	10.5	9.7
0.5 mi from Portage	4	US 25	NB			17.8	13.3	11.9	11.6	11.4	11.1	10.4	9.4
			SB			13.9	12.0	11.5	11.3	11.1	10.9	10.4	9.6
			Both	9,781	14,946ᶜ	15.5	11.7	11.2	10.9	10.7	10.6	10.0	9.3

West North Central
Iowa

Location		Route	Dir										
1.5 mi from Hinton	4	US 75	Both	5,260	7,279	11.2	10.2	9.9	9.7	9.6	9.4	—	—

Minnesota

Location		Route	Dir										
1.9 mi NW of Anoka	4	US 10	NB			27.9	24.5	22.6	21.4	20.7	19.9	16.3	—
			SB			32.2	28.6	27.5	26.6	26.0	25.2	20.6	—
			Both	11,639	25,440ᶜ	18.7	17.9	17.4	16.9	16.5	16.2	14.9	—
5.9 mi from Lakeland	4	US 12	EB			18.1	16.2	14.9	13.9	13.3	12.9	12.0	—
			WB			20.6	18.8	17.1	16.3	15.9	15.6	13.4	—
			Both	9,723	15,974ᶜ	17.7	12.9	12.6	12.2	12.0	11.8	10.9	—
2.3 mi SW of Jordan	4	US 169	NB			32.3	24.1	18.5	17.2	16.5	15.9	14.0	—
			SB			32.1	19.9	16.1	14.9	14.1	13.6	12.3	—
			Both	4,887	10,221ᶜ	22.2	18.0	16.2	14.3	13.8	13.6	12.1	—

Mountain
Idaho

Location		Route	Dir										
2.5 mi E of Post Falls	4	US 10	EB			31.6	19.5	15.8	15.0	13.9	13.2	12.1	11.2
			WB			32.4	19.6	18.0	16.8	15.8	14.8	12.7	11.0
			Both	8,567	20,862ᶜ	19.8	15.9	14.5	13.9	13.5	13.2	11.9	11.0

New Mexico

Location		Route	Dir										
4.3 mi N of Santa Fe	4	US 64	NB			21.7	14.5	13.0	12.4	12.3	12.1	11.4	10.6
		US 84	SB			21.7	18.0	16.7	16.1	15.8	15.4	14.0	—
		US 285	Both	6,718	—	21.7	14.5	13.3	12.7	12.3	12.2	11.5	10.7

For explanation of notes see Table A.1.

TABLE A.3—VARIATIONS IN TRAFFIC FLOW ON RURAL HIGHWAYS WITH MORE THAN TWO LANES

LOCATION OF COUNT STATION	NO. OF LANES	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
				AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
New England													
Maine													
3.2 mi from Biddeford	4	US 1	NB			11.8	8.5	7.5	6.4	5.9	5.4	—	—
			SB			10.5	8.2	7.4	6.4	5.8	5.5	—	—
			Both	6,966	10,878	13.1	11.9	11.4	11.2	10.9	10.7	—	—
Rhode Island													
8 mi from Providence	4	RI 146	NB			22.0	18.7	17.4	15.8	13.7	13.4	11.7	—
			SB			29.6	18.8	16.5	15.9	15.3	14.7	13.3	—
			Both	13,542	18,965	17.4	13.6	12.9	12.2	11.6	11.5	10.8	9.9
8 mi from Warwick, Rodman Hwy. S of Cranston Corners	4		NB			39.7	25.5	17.9	14.1	12.9	12.1	9.0	6.6
			SB			56.8	28.0	19.9	13.7	12.9	11.8	8.8	5.5
			Both	5,686	18,660[c]	37.0	21.5	18.2	15.8	14.1	13.3	—	—
0.5 mi from Wakefield	4	US 1	Both	5,566	19,154[c]	33.8	27.2	24.2	22.9	22.3	21.5	18.9	15.0
8 mi from Providence	4	US 6	Both	11,054	18,281[c]	13.0	12.1	11.6	11.1	10.5	10.1	—	—
Middle Atlantic													
Pennsylvania													
0.3 mi from Leetsdale	4	Pa 65	NB			11.5	9.7	9.4	9.0	8.9	8.8	—	—
			SB			10.8	10.3	10.1	10.0	9.9	9.8	—	—
			Both	15,928	20,011	10.0	9.6	9.4	9.3	9.2	9.2	8.9	8.6
South Atlantic													
Delaware													
8 mi from Wilmington	4	US 40	Both	24,927[b]	41,960[c]	12.6	11.1	10.3	10.2	9.9	9.7	—	—
5 mi from Wilmington	4	US 202	Both	14,178[b]	21,741	13.3	12.2	11.8	11.6	11.1	10.9	—	—
6 mi from Smyrna	4	US 13	Both	12,516[b]	22,451[c]	15.2	14.4	13.8	13.3	12.7	12.5	—	—
2 mi from Milford	4	US 113	Both	6,911[b]	14,757[c]	18.9	17.1	16.2	15.9	15.3	15.0	—	—
1 mi S of Greenwood	4	US 13	Both	6,142[b]	10,543[c]	12.0	10.6	10.2	10.0	9.8	9.7	—	—
Florida													
15 mi S of Jacksonville	4	US 1	NB			35.2	14.3	13.0	11.7	11.3	11.1	10.4	—
			SB			15.9	12.1	11.1	10.7	10.5	10.4	9.6	—
			Both	10,525	—								

Location	Lanes	Route	Dir											
0.5 mi S of Florida City	4	US 1	NB			24.9	23.1	21.4	19.7	18.8	17.5	15.8	—
			SB			19.2	16.2	14.9	14.5	14.1	13.6	12.3	—
			Both	4,625	—								
2.5 mi S of Oak Hill	4	US 1	NB			15.9	13.9	13.3	13.0	12.3	12.0	11.0	—
			SB			29.9	15.2	14.2	13.3	13.0	12.6	11.4	—
			Both	6,245	—								
North Carolina													
1 mi from Wrightsville Beach on Wrightsville Beach Bridge	4		Both	6,520	16,732ᶜ	24.4	23.2	21.4	20.6	19.8	18.8	16.5	—
South Carolina													
4.3 mi E of Easley	4	US 123	Both	9,462	14,735ᶜ	14.8	11.9	10.1	9.7	9.5	9.3	8.9	8.4
East North Central													
Indiana													
5.5 mi from Greenfield	4	US 40	EB			26.9	11.4	11.7	11.0	10.9	10.5	—	—
			WB			—	—	—	—	—	—	—	—
			Both	8,791	13,592ᶜ								
8.5 mi from Columbus	4	US 31	NB			16.0	14.5	14.4	14.2	13.7	11.6	—	—
			SB			—	—	—	—	—	—	—	—
			Both	10,504	16,703ᶜ								
15.2 mi from Lafayette	4	US 52	EB			17.6	13.9	14.9	11.5	11.3	10.5	—	—
			WB			—	—	—	—	—	—	—	—
			Both	8,488	13,077ᶜ								
Michigan													
2 mi from Mt. Clemens	4	US 25	NB			16.0	15.0	14.4	14.1	13.9	13.7	12.5	11.0
			SB			21.9	19.8	19.3	18.2	18.0	17.5	15.4	11.7
			Both	15,518	28,501ᶜ	13.8	12.9	12.6	12.4	12.2	11.9	11.3	10.3
2.5 mi from St. Johns	4	US 27	NB			27.4	21.1	19.4	18.7	18.0	17.5	15.8	13.0
			SB			39.8	29.3	28.1	26.9	25.8	25.1	20.7	14.3
			Both	8,252	18,017ᶜ	22.3	18.4	17.6	17.2	16.8	16.2	14.9	13.1
5 mi from Drayton Plains	4	US 10	NB			18.1	14.3	13.9	13.3	12.9	12.5	11.4	10.6
			SB			22.0	20.8	19.3	18.4	17.9	17.4	15.0	12.0
			Both	9,226	18,069ᶜ	16.8	15.5	14.8	14.4	14.0	13.7	12.4	10.9
7.7 mi from Monroe	4	US 24	NB			30.8	16.3	14.7	13.4	12.4	11.3	10.2	9.3
			SB			26.8	15.3	13.6	12.5	11.8	11.3	10.2	9.1
			Both	4,441	6,809ᶜ	17.5	12.2	11.4	10.9	10.4	10.2	9.6	8.7
2.8 mi from Schoolcraft	4	US 131	NB			15.1	12.2	11.6	11.3	11.2	10.9	10.2	9.4
			SB			17.5	12.3	11.8	11.3	11.1	10.8	10.1	9.3
			Both	5,446	7,982ᶜ	12.9	11.5	11.3	11.0	10.8	10.7	10.0	9.3
Wisconsin													
9.0 mi from Fond DuLac	4	US 41	NB			33.3	23.2	21.9	21.2	20.6	20.1	17.6	13.3
			SB			34.8	27.2	25.2	24.6	23.9	23.3	20.7	15.9
			Both	9,144	21,063ᶜ	21.1	18.5	17.2	16.9	16.2	15.9	14.6	13.4

TABLE A.3—VARIATIONS IN TRAFFIC FLOW ON RURAL HIGHWAYS WITH MORE THAN TWO LANES (CONT.)

LOCATION OF COUNT STATION	NO. OF LANES	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
				AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
West North Central													
Missouri													
20 mi from Kansas City	4	Rt 69	SB			18.7	17.0	16.5	15.9	15.4	15.1	14.0	12.7
			Both	8,544	13,318c								
11 mi from Warrensburg	4	Rt 50	EB			29.8	26.7	24.3	22.8	21.9	21.3	17.9	14.3
			Both	4,459	10,312								
15 mi from St. Louis	4	Rt 61	Both	15,342	25,411	11.8	11.2	11.0	10.7	10.4	10.2	9.7	9.1
East South Central													
Alabama													
0.5 mi from Morris	4	US 31	Both	8,180	12,957c	13.1	11.6	11.3	11.1	10.9	10.7	9.9	9.1
Tennessee													
1 mi N of Goodlettsville	4	SR 11	NB			10.9	10.2	9.7	10.1	10.1	9.5	9.3	8.9
			SB			13.9	12.7	11.9	10.0	10.9	10.7	9.5	8.9
			Both	13,662	21,095c								
West South Central													
Louisiana													
6.2 mi W of Port Allen	4	US 190	EB			22.8	16.8	16.0	15.3	14.8	14.4	—	—
			WB			17.7	12.4	11.3	11.1	10.8	10.6	—	—
			Both	12,306	19,129c	17.2	13.0	12.4	12.0	11.7	11.5	—	—
3 mi from Norco	4	US 61	EB			22.4	15.6	14.6	14.1	13.8	13.3	8.8	7.8
			WB			16.5	12.9	11.3	11.0	10.7	10.4	8.8	8.1
			Both	11,981	—	13.6	11.8	11.4	11.1	10.7	10.4	8.7	7.5
1.9 mi SW of Denham Springs	4	US 190	EB			17.2	12.8	12.1	11.7	11.4	11.3	10.3	8.2
			WB			14.3	12.0	11.3	11.1	10.9	10.8	10.1	8.8
			Both	11,307	15,303c	11.2	10.0	9.6	9.4	9.3	9.2	8.3	7.8
8 mi E of Shreveport	4	US 80	EB			15.1	11.3	10.9	10.5	10.3	10.2	9.1	8.4
			WB			19.0	12.5	11.7	11.2	10.9	10.6	9.2	8.6
			Both	11,530	16,611c	12.8	10.3	10.0	9.8	9.7	9.5	—	—
Texas													
4.3 mi E of Shamrock	4	US 66	EB			15.9	13.9	13.0	12.0	11.5	11.2	10.3	9.6
			WB			17.7	14.3	13.5	13.2	12.7	12.6	11.8	11.0
			Both	4,170	9,305c								

Mountain													
Arizona													
2 mi from Wickenburg	4	US 60	Both	5,354	11,659c	13.2	8.6	8.1	7.9	7.8	7.8	7.8	7.7
Colorado													
3 mi E of La Junta	4	US 50	EB	.		17.1	15.7	14.7	13.9	13.7	13.5	12.1	11.1
			WB			16.0	15.5	14.9	14.4	13.9	13.3	11.8	10.9
			Both	2,497	4,588c	14.4	13.9	13.3	12.5	12.0	11.9	11.4	10.6
Nevada													
3 mi from Verdi	4	US 40	EB			18.5	15.3	14.5	13.7	13.4	13.2	12.4	11.3
			WB			20.2	17.1	16.1	15.6	15.2	14.7	13.1	11.5
			Both	6,279	13,514c	14.0	13.0	12.7	12.5	12.3	12.2	11.5	10.5
1.5 mi from Sparks	4	US 40	EB			13.3	11.6	10.9	10.5	10.2	10.0	9.5	8.9
			WB			13.9	12.7	12.1	11.6	11.4	11.3	10.5	9.9
			Both	5,820	9,522c	11.1	10.5	10.2	10.1	9.9	9.8	9.4	8.9
Utah													
1 mi from Farmington	4	US 91 & US 89	Both	19,361	27,075c	12.0	11.4	11.0	10.8	10.5	10.4	—	—
4.5 mi from Murray	4	US 91, US 89 & US 50	Both	12,146	22,737c	15.7	12.8	12.2	11.8	11.3	11.0	—	—
6 mi from S Salt Lake	4	US 40	Both	6,114	18,978c	30.5	24.6	22.1	21.5	20.9	20.2	—	—
Pacific													
Oregon													
Pacific Highway, N. of Ashland	4	US 99	Both	10,676	17,580c	11.0	10.5	10.1	9.9	9.8	9.7	9.1	8.6
Washington													
SE leg of jct; in Seattle	4	US 99 & 216 St.	NB			12.3	11.4	11.0	10.6	10.4	10.1	9.3	8.5
			Both	32,276	48,040c								

For explanation of notes see Table A.1.

TABLE A.4—VARIATIONS IN TRAFFIC FLOW ON TWO-LANE RURAL HIGHWAYS

LOCATION OF COUNT STATION	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
			AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
New England												
Connecticut												
1.0 mi N of Conn 163, Montville	Conn 32	Both	8,900	11,272	12.2	10.2	10.1	10.0	10.0	9.9	9.6	9.1
S of US 62, Woodbury	US 6	Both	5,600[b]	9,549[c]	18.1	16.0	15.1	14.2	13.7	13.2	12.1	10.3
S of Route 80, Clinton	Conn 81	Both	1,500[b]	5,477[c]	29.9	27.8	26.1	24.7	23.9	23.3	20.7	17.7
Maine												
2.0 mi from Farmington	US 2	Both	3,827	7,143	17.2	14.9	13.5	12.7	12.3	12.0	—	—
2.7 mi from Houlton	US 1	Both	3,274	4,545	11.9	10.8	10.3	10.0	9.9	9.7	—	—
5.5 mi from Augusta	US 201	Both	3,035	4,221	12.5	11.0	10.1	9.9	9.7	9.6	—	—
New Hampshire												
8 mi from Portsmouth	US 1	Both	7,091	11,960[c]	14.2	13.1	12.7	12.4	—	—	—	—
Vermont												
2 mi from Rutland	US 7	Both	6,462	11,970[c]	15.1	13.2	12.6	12.4	12.1	12.0	11.3	—
Middle Atlantic												
New Jersey												
South River Rd., E. Brunswick Twp.		Both	4,145[b]	—	16.5	13.3	12.7	11.2	10.8	10.5	10.3	9.8
Elmer Cantreton Rd., 16.0 mi from Bridgeton		Both	2,092[b]	—	14.6	13.7	12.9	12.7	12.1	12.0	11.3	10.2
New York												
8.0 mi from Livonia	US 15	Both	8,000[b]	—	30.3	18.5	17.8	17.1	16.4	15.9	13.9	11.9
0.05 mi W of W. Winfield Village	US 20	Both	2,800	—	26.4	19.6	18.2	16.4	15.0	15.0	12.5	10.7
Pennsylvania												
1.0 mi from Tinleyville	Pa 88	Both	3,848	6,823[c]	16.7	12.0	11.1	10.8	10.5	10.3	9.8	9.3
South Atlantic												
Delaware												
4 mi SW of Milford, Sussex Co.	SR 36	Both	1,473[b]	2,130[c]	21.6	11.3	10.6	10.3	10.2	10.0	—	—
Maryland												
3.5 mi from Hagerstown	US 40	Both	9,680	—	14.5	12.3	11.7	11.3	—	10.8	—	—
Virginia												
0.9 mi W of Richmond (W city limits)	Rt 6	Both	11,030	14,851	16.4	14.1	13.6	13.1	12.7	12.5	11.5	10.4
11.1 mi from Bowling Green	Rt 301	Both	7,623	21,587[c]	15.1	14.5	14.2	13.5	13.3	13.1	11.1	9.8
0.3 mi SE of Fredericksburg	Rt 2 & 17	Both	7,080	14,110[c]	16.1	13.3	12.4	11.7	11.6	11.3	10.0	9.3

6.6 mi from Lynchburg	Rt 291	Both	6,160	8,985ᶜ	12.8	11.3	10.9	10.6	10.4	10.2	9.8	9.3
1.0 mi NW of Tappahannock	Rt 17	Both	3,158	6,112ᶜ	17.1	15.0	14.5	13.5	13.5	13.1	12.4	11.3
1.3 mi E of Tallysville	Rt 33	Both	1,854	3,760ᶜ	23.5	20.9	19.5	18.8	17.9	17.5	14.5	11.8
1.3 mi S of Fincastle (S city limit)	Rt 220	Both	2,808	4,127ᶜ	15.5	13.5	12.2	11.9	11.7	11.5	10.5	9.7
0.2 mi from Standardsville	Rt 33	Both	1,319	3,395ᶜ	35.9	23.0	21.1	19.9	18.0	17.4	14.8	12.3
1.7 mi W of Rt 3	Rt 20	Both	974	2,730	36.1	23.0	20.9	19.0	18.5	18.0	15.9	13.8
12.2 mi from Farmville	Rt 45	Both	682	1,244ᶜ	22.1	15.2	14.5	14.1	13.8	13.6	12.3	11.0
2.0 mi S of Steeles Tavern	Rt 56	Both	200	490ᶜ	42.0	29.0	28.5	22.5	21.5	21.0	17.5	14.5
4.0 mi S of Rt 60	Rt 156	Both	198	394ᶜ	35.4	26.3	24.7	21.7	19.7	19.2	16.7	13.6
West Virginia												
1.9 mi from Triadelphia	US 40	Both	8,886	15,120ᶜ	12.0	10.4	9.9	9.8	9.6	9.4	—	—
2.1 mi from Martinsburg	US 11	Both	7,569	10,726ᶜ	11.5	10.1	9.5	9.2	9.1	9.0	—	—
3.1 mi from Huntington	US 52	Both	4,558	6,652ᶜ	12.3	11.2	10.7	10.4	10.2	10.2	—	—
4.8 mi from Henderson	SH 17	Both	2,889	7,571ᶜ	22.1	15.0	12.9	11.9	11.5	11.1	—	—
2.9 mi from Welch	US 52	Both	3,560	4,786ᶜ	11.2	9.8	9.5	9.4	9.2	9.2	—	—
Florida												
4.0 mi S of Punta Garda	SR 45	Both	4,795	—	14.3	13.0	12.6	12.4	12.0	11.8	11.4	—
4.0 mi E of Crestview	SR 10	Both	2,940	—	26.4	12.4	11.4	11.1	11.1	10.4	9.7	—
Georgia												
6 mi W of Athens	US 29 & 78	Both	9,202	16,101ᶜ	22.0	11.9	10.3	9.8	9.7	9.5	—	—
3 mi NE of Statesboro	US 301	Both	6,136	10,584ᶜ	14.0	11.5	11.0	10.6	10.2	9.9	—	—
1 mi SW of Midway	US 17	Both	5,301	8,279ᶜ	11.3	10.6	10.3	10.9	9.8	9.6	—	—
6.5 mi SE of Thomson	US 78	Both	4,424	6,759ᶜ	16.4	11.2	10.6	10.2	9.9	9.6	—	—
6 mi NE of Gainesville	US 23	Both	4,261	6,459ᶜ	14.0	12.0	11.4	10.6	10.1	9.7	—	—
North Carolina												
1.8 mi S of Halifax city limits	US 301	Both	7,240	17,397ᶜ	17.0	14.5	13.0	12.2	11.9	11.7	10.8	—
8 mi from Asheboro	US 220	Both	8,100	10,668ᶜ	12.3	11.1	10.7	10.2	10.1	9.9	8.7	—
South Carolina												
4.9 mi S of Hardeeville	US 17	Both	5,722	10,410ᶜ	13.3	11.3	10.7	10.2	9.9	9.8	9.2	8.7
4.3 mi SW of Manning	US 301	Both	4,351	9,532ᶜ	17.1	15.0	14.0	13.0	12.4	12.1	11.1	10.0
10.0 mi S of Greenville	US 25	Both	3,776	5,546	12.4	11.4	10.8	10.4	10.1	10.0	9.3	8.7
5.5 mi W of Lexington	US 1	Both	3,586	5,321ᶜ	12.4	10.9	10.5	10.0	9.8	9.6	9.1	8.4
1.2 mi S of Society Hill	US 15	Both	3,498	6,906ᶜ	22.6	11.9	10.1	9.7	9.3	9.1	8.4	7.8
0.1 mi S of SC 97, Chester	US 321	Both	1,935	3,080ᶜ	14.1	11.6	10.9	10.5	10.2	9.8	9.0	8.0
1.1 mi SE of Rosinville	US 178	Both	1,244	2,190ᶜ	21.3	16.8	15.6	14.0	13.4	12.7	11.4	9.8
East North Central												
Illinois												
NE of Edwardsville	Bypass US 66	Both	3,700	6,900ᶜ	12.8	11.4	10.9	10.5	10.2	10.1	9.5	8.8
2.2 mi E of Mendota	US 34	Both	2,500	3,978	14.0	13.2	12.5	12.0	11.6	11.4	10.4	9.6
5.5 mi from London Mills	Ill 166	Both	1,550	2,610ᶜ	19.0	14.6	13.8	13.2	12.8	12.5	11.4	10.3

LOCATION OF COUNT STATION	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
			AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
Illinois (Cont'd)												
City of Watson	US 45	Both	1,500	3,041c	19.0	15.7	13.4	12.6	12.2	11.9	10.6	9.6
East of Allerton	FAS 514	Both	746	1,097c	17.8	12.2	11.1	10.9	10.7	10.6	10.2	9.7
Indiana												
3 mi from Vincennes	US 41	Both	7,542	11,499c	10.6	10.1	9.7	9.5	9.3	9.0	—	—
6 mi from Paragon	SR67 & 6	Both	5,152	9,056	16.4	14.2	13.5	12.8	12.2	12.0	—	—
6.5 mi from North Webster	SR 13	Both	2,572	769c	29.9	27.5	26.4	25.2	23.8	22.2	—	—
1 mi from Argos	SR 31	Both	4,610	6,277	16.7	11.3	10.7	10.2	10.0	9.9	—	—
7.8 mi E Wappanee	US 6	Both	3,947	5,898c	12.4	11.4	10.5	10.2	9.9	9.6	—	—
2.1 mi from Rome City	SR 9	Both	2,695	4,999c	16.7	15.4	14.7	14.3	13.7	13.0	—	—
Michigan												
1.4 mi N of Jct. US 16	US 23	Both	7,533	18,888c	18.5	17.9	17.7	17.1	16.8	16.5	15.6	14.2
2.0 mi from Standish	M 76	Both	3,583	11,819c	34.4	29.4	27.7	26.4	25.1	24.0	21.0	17.7
4.5 mi S of Main Four Corners, Wolverine	US 27	Both	2,915	11,550	38.2	34.1	31.6	30.2	28.8	27.7	24.0	20.4
2.7 mi S of Traverse City city limits	US 31 & M 37	Both	3,849	9,206	22.8	16.9	16.4	15.9	15.5	15.3	13.7	12.7
3.5 mi N of W city limit of Morley	US 131	Both	3,902	7,269c	20.3	17.7	16.6	15.1	14.7	14.5	13.4	12.0
2.0 mi S of 4th St. in Baldwin	M 37	Both	2,722	7,714c	27.3	23.9	21.4	20.5	20.1	19.8	18.1	16.1
10 mi from Tarwell	M 115	Both	1,922	6,357c	32.8	29.4	27.2	25.9	25.0	23.9	20.6	17.1
2.4 mi N of Rose City N of city limit	M 33	Both	1,689	5,312c	60.0	38.3	32.1	29.2	27.9	26.5	22.9	19.0
1.7 mi E of Brevort	US 2	Both	1,877	8,600	30.0	28.5	26.6	25.9	24.3	24.0	21.9	19.4
1.0 mi E of E city limit of Buchanan	Co. Rd.	Both	3,626	4,883	13.6	12.4	12.1	11.9	11.7	11.6	11.0	10.5
4.6 mi from Pentwater	US 31	Both	2,292	5,561c	22.3	19.9	19.0	18.6	18.0	17.3	15.3	13.1
5.0 mi from Port Sanilac	US 25	Both	1,451	4,768	38.5	31.0	27.9	26.6	25.2	24.3	20.6	16.6
3 mi from Capac	M 21	Both	2,857	5,478	15.1	14.3	13.6	13.3	13.0	12.7	12.0	10.9
3.2 mi S of Alpena S city limit	US 23	Both	2,872	5,263c	16.1	13.7	12.8	12.4	12.2	12.0	11.5	10.8
0.1 mi W of Jct. M 95—US 41—M 28		Both	2,242	5,113c	21.9	17.1	16.1	15.5	15.0	14.6	13.8	12.6
0.3 mi from Hermans	M 53	Both	1,604	3,981c	34.2	24.1	22.2	20.8	19.4	18.5	15.7	13.5
6.8 mi N of N jct. with M 72	US 131	Both	1,607	4,132c	25.0	20.7	19.7	18.6	18.0	17.6	15.8	13.8
1.2 mi E of Jct. M 99	US 12	Both	2,838	4,120c	13.0	10.9	10.5	10.3	10.1	9.9	9.3	8.6
0.7 mi W of W city limit of Homer	M 60	Both	2,858	3,697	26.0	10.9	10.3	10.1	9.9	9.7	9.0	8.4

0.5 mi from Pompeii	M 57	Both	1,586	3,151c	20.2	18.3	17.3	16.2	15.9	15.3	12.9	11.2
0.4 mi E of Raco Corners	M 28	Both	1,132	2,779	24.2	21.7	20.6	20.1	19.6	19.3	18.2	16.9
0.5 mi S of US 10	M 66	Both	979	2,944c	31.9	28.4	23.8	22.4	21.2	20.4	18.3	14.9
6.8 mi N of Skandia	US 41	Both	1,703	3,558c	18.3	14.3	13.6	12.7	12.0	11.7	11.3	10.3
9.8 mi W of W city limit of Ithaca	Co. Rd.	Both	1,054	3,205c	26.0	19.9	17.9	16.8	16.0	15.2	13.2	11.1
2.8 mi N of N city limit of Zeeland	Co. Rd.	Both	1,315	1,890c	26.0	12.9	12.2	12.0	11.8	11.6	11.1	10.4
10.2 mi S of jct. with US 2	US 41	Both	1,151	2,009	17.8	14.7	14.0	13.2	12.7	12.2	10.9	9.8
1.3 mi E of E city limit of Marshall	Co. Rd.	Both	1,339	1,896	21.0	12.1	11.8	11.4	11.1	11.0	10.4	9.9
10 mi W of Iron River		Both	904	1,919c	18.7	17.4	17.0	16.5	16.3	16.2	15.2	13.9
3.3 mi N of N city limit of Lake Odessa	Co. Rd.	Both	796	2,322c	29.8	19.2	16.1	14.8	13.6	13.1	11.7	10.7
5.5 mi from Houghton Lake	Co. Rd.	Both	368	1,658c	63.9	38.6	34.0	30.4	29.1	28.0	24.2	19.0
10 mi from Cascada	Co. Rd.	Both	707	1,333	25.6	16.4	15.7	14.7	14.1	13.9	12.7	11.7
5.1 mi N of Paw Paw St. in Lawrence	Co. Rd.	Both	612	1,257c	20.6	18.3	17.2	16.3	16.0	15.7	14.2	12.7
Wisconsin												
3.7 mi from Menomonee Falls	SH 175	Both	7,521	9,806	12.9	11.5	11.1	11.0	10.9	10.8	10.5	10.1
6.0 mi from Kaukauna	US 41	Both	5,341	9,526	26.1	21.4	13.6	12.1	11.2	10.7	10.0	9.4
West North Central												
Iowa												
NW leg Jackson & US 218, Charles City	US 218	Both	7,153	10,556c	12.4	11.2	10.6	10.1	9.8	9.7	—	—
1 mi S of Ames city limits	US 69	Both	6,585	9,524c	12.8	11.4	11.0	10.5	10.4	10.2	—	—
0.5 mi E of Lean city limits	SH 2	Both	1,488	2,340	16.5	15.1	14.3	13.4	13.0	12.6	—	—
Kansas												
2 mi N of McPherson city limits	US 81	Both	4,237	7,458c	15.6	12.4	11.1	10.9	10.8	10.6	9.7	—
2.5 mi from Leon city limits	K 96	Both	2,551	5,189c	30.5	19.1	18.0	17.0	16.2	15.7	14.1	—
2.5 mi E of Great Bend, Banton Co.	US 56	Both	3,840	6,450	15.1	11.9	10.7	10.2	9.9	9.7	9.1	—
4 mi from Tonganoxie	US 24	Both	3,246	5,030	13.1	12.3	12.0	11.9	11.6	11.4	11.1	—
2.8 mi from Fort Scott city limits	US 69	Both	2,760	4,156c	17.4	13.9	13.2	12.8	12.4	12.3	11.2	—
5.8 mi W of Kingman city limits	US 54	Both	3,072	5,945c	19.4	13.7	11.9	11.2	10.7	10.6	10.0	—
3.0 mi NE of Viola, Sedgwick Co.	K 42	Both	2,261	3,815c	19.4	15.7	14.6	14.2	13.8	13.4	12.6	—
3.7 mi N of Arkansas city limits	US 77	Both	2,601	3,375	14.0	12.6	12.3	11.9	11.7	11.5	10.9	—
1.0 mi NW of Rossville	US 24	Both	2,309	3,615c	18.8	14.4	14.0	13.1	12.4	12.0	10.9	—
6.3 mi from Holton	US 75	Both	2,195	3,605c	17.9	14.0	12.9	12.6	12.1	11.9	10.8	—
3 mi from Colby	US 24	Both	1,970	4,318c	17.1	14.3	13.5	13.1	12.9	12.6	11.7	—
4 mi from Parsons	US 160	Both	2,055	2,811	13.6	12.8	12.2	11.8	11.4	11.1	10.0	—
4.5 mi from Iola	US 54	Both	2,102	3,437	15.3	12.8	11.7	11.4	11.0	10.9	10.0	—
3.5 mi from Belleville	US 81	Both	2,010	3,700c	17.6	12.5	11.7	11.4	10.9	10.6	9.4	—
1.0 mi W of Wilson, Russell Co.	US 40	Both	1,859	3,692c	21.5	14.0	12.5	11.9	11.6	11.2	10.5	—
1.5 mi from Yates Center	US 75	Both	1,830	2,836	17.7	13.6	12.6	12.1	11.9	11.6	10.7	—
7 mi from St. Johns	US 281	Both	1,747	2,969c	16.8	13.1	12.0	11.6	11.1	10.6	9.8	—
3 mi from Phillipsburg	US 183	Both	1,537	2,847c	24.0	14.0	12.1	11.5	11.1	10.7	10.2	—
5.5 mi from Wellington	US 81	Both	1,555	2,579c	13.0	11.9	11.6	11.3	11.1	10.9	10.3	—

LOCATION OF COUNT STATION	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
			AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
Kansas (Cont'd)												
1.5 mi from Lincoln	K 18	Both	1,464	3,102	23.6	15.4	13.6	11.9	11.5	10.7	10.0	—
6 mi from Arlington	K 61	Both	1,305	2,882c	17.9	14.6	12.5	11.8	11.3	10.9	9.6	—
3 mi from Oakley	US 40	Both	1,203	2,419c	14.9	13.6	13.1	12.8	12.5	12.3	11.5	—
4.6 mi from Clay Center	K 15	Both	1,168	1,705c	15.5	12.8	12.0	11.6	11.4	11.2	10.5	—
12 mi W of Scott city limits	K 96	Both	1,072	1,922c	16.3	13.5	12.5	11.9	11.6	11.3	10.5	—
8 mi from Harper	US 160	Both	992	1,593	15.7	14.5	12.8	12.2	11.8	11.3	10.4	—
7 mi from Oakley	US 83	Both	764	1,676c	24.2	16.8	15.8	15.2	14.5	14.1	11.8	—
2 mi from Greeley	US 169	Both	828	2,156c	32.6	15.1	13.4	12.7	12.4	12.3	11.0	—
3.5 mi from Hill City	US 283	Both	728	1,186c	22.4	14.0	13.1	12.4	12.0	11.7	11.1	—
3.5 mi from Ulysses	K 25	Both	725	1,140c	22.3	13.0	12.0	11.7	11.3	11.2	10.5	—
11 mi from Kingman	FAS 303	Both	385	1,021	42.6	21.3	18.4	16.6	15.3	14.3	12.7	—
7.5 mi from Dorrance	FAS 591	Both	143	379c	47.6	31.5	29.4	26.6	25.2	24.5	20.3	—
0.8 mi from Mankato	FAS 340	Both	183	298	35.5	19.1	16.9	15.8	15.3	14.8	13.7	—
4.9 mi from Shakopee	US 212	Both	7,736	13,386c	16.5	14.3	13.2	12.5	12.3	11.9	10.9	—
4.8 NW of Lakeville	US 65	Both	7,262	13,148c	17.2	15.0	13.4	12.8	12.6	12.3	11.4	—
South Dakota												
11 mi from Rapid City	US 14 & 16	Both	3,169	6,048	16.3	14.0	13.6	13.2	—	—	—	—
6.5 mi from Sioux Falls	US 77	Both	3,254	4,729c	14.8	12.2	11.4	11.0	—	—	—	—
4 mi from Huron	US 14	Both	2,392	5,961	22.1	15.9	13.4	12.7	—	—	—	—
5 mi from Aberdeen	US 12	Both	2,678	4,884	16.6	12.8	11.5	10.8	—	—	—	—
6.4 mi from White Lake	US 16	Both	1,454	2,682c	47.8	16.7	14.7	14.0	—	—	—	—
2.5 mi from Watertown	US 81	Both	1,462	2,585	16.8	12.9	11.5	11.2	—	—	—	—
1.2 mi from Bonesteel	US 18	Both	841b	1,392c	18.3	13.0	11.8	11.2	—	—	—	—
5.5 mi SE of Wall city limits	I 90	Both	823	1,747c	18.1	14.0	13.2	12.5	—	—	—	—
Nebraska												
6 mi from Fremont	US 77	Both	5,124	8,225c	16.0	12.4	11.7	11.5	11.1	10.9	10.0	9.0
5 mi from Elm City	US 30	Both	4,187	8,078c	14.9	12.8	12.2	12.0	11.8	11.7	11.2	10.7
4 mi W of S. Sioux City	US 20	Both	3,358	5,733c	15.4	12.1	11.7	11.4	11.1	11.0	10.2	9.0
North Dakota												
9 mi from Minot	US 83	Both	4,803	7,355	19.4	16.8	16.3	15.8	14.8	14.6	—	—

Location	Route	Dir										
3 mi from Sterling	US 10	Both	2,653	4,528c	15.6	12.9	12.1	11.8	11.6	11.4	—	—
5.5 mi from Belfield	US 10	Both	1,081	3,351c	33.9	21.6	20.0	19.2	18.1	17.8	—	—
0.5 mi from Michigan City	US 2	Both	1,196	2,209c	18.6	14.4	13.7	13.3	12.9	12.6	—	—

East South Central
Alabama

Location	Route	Dir										
2.6 mi E of Barton city limits, Colbert Co.	US 72	Both	5,064	6,976	15.5	14.7	14.5	14.2	13.9	13.8	13.5	12.6
7.5 mi from Montgomery City	US 82 & US 231	Both	5,982	10,617	13.1	11.4	10.8	10.1	10.0	9.8	9.1	8.4
11 mi S of Mobile at Dog River	SH 63	Both	4,536	7,532c	17.2	14.4	13.6	13.1	12.7	12.4	11.2	10.0
2 mi W of Riverside, St. Clair Co.	US 78	Both	5,154b	7,847c	18.4	12.2	11.5	11.2	10.9	10.7	9.8	8.7
3.5 mi NE of Loxley, Baldwin Co.	US 90	Both	5,281	8,108c	12.9	11.2	10.8	10.6	10.4	10.3	9.8	9.1
1.5 mi from Madison, Madison Co.	US 72A & SH 20	Both	4,851	6,637	12.5	11.5	10.9	10.6	10.5	12.3	9.7	9.1
0.5 mi SW of Mt. Vernon, Mobile Co.	US 43	Both	3,520	4,863c	20.5	12.0	11.3	11.1	10.9	10.6	9.7	9.1
1.2 mi from Paint Rock, Jackson Co.	US 72	Both	2,638	5,081c	15.1	13.9	13.1	12.7	12.3	12.2	11.4	10.2

Kentucky

Location	Route	Dir										
1.5 mi SE of Mt. Vernon	US 25	Both	5,734	—	16.7	14.3	13.1	12.6	12.4	12.0	—	—
1 mi S of Crofton	US 41	Both	4,190	—	20.8	12.2	11.0	10.3	9.8	9.5	—	—
1 mi NW of Flemingsburg	Ky 11	Both	2,533	—	23.7	17.0	15.4	14.6	14.2	13.8	—	—
2 mi SE of Sturgis	US 60	Both	3,225	—	16.1	12.7	12.0	11.4	10.9	10.5	—	—
4 mi N of Pikeville	US 23	Both	3,500	—	12.6	11.4	10.6	10.3	10.3	10.0	—	—
2 mi W of Grayson	US 60	Both	3,103	—	13.5	12.2	11.9	11.3	11.0	11.0	—	—
8 mi from Frankfort	US 60	Both	3,058	—	14.4	12.4	11.8	11.4	11.4	11.1	—	—
3 mi E of Caneyville	US 62	Both	2,047	—	16.6	14.7	14.2	13.7	13.2	13.2	—	—
0.8 mi W of Loyal	US 119	Both	2,596	—	13.8	10.5	10.1	10.0	9.8	9.6	—	—
4 mi N of Dry Ridge	US 25	Both	1,930	—	15.9	14.0	14.0	13.5	13.0	13.0	—	—
1 mi SW of Wingo	US 45	Both	2,001	—	13.0	11.0	10.5	10.5	10.0	9.5	—	—
2 mi W of Campton	Ky 15	Both	1,429	—	16.3	14.7	13.9	13.3	13.3	12.6	—	—
1.8 mi SE of Shelbyville, Old Taylorsville Rd.		Both	364	—	24.7	14.0	13.7	12.9	12.1	11.8	—	—

Mississippi

Location	Route	Dir										
6 mi W of W city limit of Bay St. Louis	US 90	Both	6,520	12,038c	16.2	14.7	14.2	13.7	13.4	13.1	11.7	10.4
1.5 mi E of E city limit of Bolton	US 80	Both	4,656	7,989	11.1	10.4	9.9	9.7	9.6	9.4	9.0	8.1
1.5 mi N of N city limit of Madison	US 51	Both	4,279	5,621	11.4	10.4	10.2	10.0	9.9	9.6	9.1	8.5
7 mi from Columbus	US 82	Both	3,131	6,056c	22.5	11.9	10.5	10.1	9.8	9.5	8.9	8.3
7 mi from Vicksburg	US 61	Both	2,612	3,579c	17.3	13.0	12.1	11.5	11.4	11.1	10.2	9.2

Tennessee

Location	Route	Dir										
3.6 mi SW of Brownsville	US 70	Both	4,547	7,347c	17.8	14.5	13.5	13.0	12.5	12.1	10.4	8.8
2.5 mi from Murfreesboro	US 41	Both	5,138	9,126c	12.9	11.3	11.0	10.7	10.5	10.3	9.5	7.8
11.5 mi from Madisonville	US 411	Both	3,902	6,646c	17.9	13.8	13.0	12.5	12.1	11.8	10.2	9.5
2.7 mi from Dayton	US 27	Both	4,291	7,458	15.2	10.6	10.1	9.8	9.7	9.5	8.9	8.0

TABLE A.4—VARIATIONS IN TRAFFIC FLOW ON TWO-LANE RURAL HIGHWAYS (CONT.)

LOCATION OF COUNT STATION	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
			AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
West South Central												
Arkansas												
0.5 mi from Forrest City	US 70	Both	10,125	11,800c	9.4	8.8	8.5	8.4	8.3	8.2	—	—
2.6 mi from Hazen	US 70	Both	4,569	6,186c	11.8	10.6	10.1	9.8	9.6	9.5	—	—
6 mi SW of Prescott	US 67	Both	4,250	6,150c	13.7	11.4	10.8	10.4	10.1	9.9	—	—
2.4 mi from Wilson	US 61	Both	4,800	6,125c	13.3	9.6	9.2	9.0	8.8	8.7	—	—
5.5 mi from Marrilton	US 64	Both	4,025	5,200c	15.0	11.6	11.3	10.6	10.5	10.3	9.5	—
Louisiana												
1.0 mi S of Plaquemine	La 1	Both	3,541	4,586	13.4	11.2	10.8	10.3	10.1	9.9	8.9	8.3
Oklahoma												
3 mi from Duncan	US 81	Both	5,938	—	12.3	10.4	10.0	9.6	9.3	9.2	8.7	—
2 mi N of Davis city limits	US 77	Both	4,306	—	18.5	15.2	13.8	12.8	12.1	11.8	9.8	—
4 mi W of Stillwater	SH 51	Both	3,324	—	21.8	19.0	17.1	15.1	14.1	13.3	10.4	—
3 mi W of Vinita	US 66	Both	4,650	—	12.5	10.8	10.3	10.0	9.8	9.6	8.7	6.9
6.5 mi NE of Durant	US 69	Both	3,703	—	14.7	11.9	11.4	10.0	9.5	9.1	7.8	—
3 mi E of Keystone	US 64	Both	3,026	—	15.5	13.2	12.0	11.6	10.9	10.6	9.1	—
12 mi from Enid	US 81	Both	3,014	—	13.6	12.0	11.7	11.4	11.0	10.7	9.7	7.5
1.5 mi from Colgate	US 75	Both	2,733	—	21.2	15.1	12.1	11.6	10.6	10.3	8.9	—
2 mi SW of Woodward	US 183	Both	2,424	—	13.9	12.0	10.7	10.4	10.1	9.9	9.2	—
3.5 mi from Hobart	US 183	Both	2,237	—	16.9	12.3	11.7	11.0	10.9	10.5	9.4	—
8.5 mi from McAlester	US 270	Both	1,642	—	24.7	14.3	12.7	11.9	11.5	11.3	10.1	—
6 mi E of Wetumka	SH 9	Both	1,150	—	21.9	16.7	15.6	15.0	14.5	13.9	12.3	9.0
Texas												
3.0 mi from Humble	US 59	Both	8,190	13,155c	15.9	14.3	13.7	13.3	12.7	12.6	11.4	10.2
1.0 mi E of Cypress, Harris Co.	US 290	Both	6,200	10,900c	18.3	15.1	14.2	13.4	12.9	12.8	11.0	9.5
Mountain												
Arizona												
11 mi from Flagstaff	US 66	Both	4,440	10,496c	15.7	11.3	10.9	10.8	10.8	10.7	10.7	10.6
7 mi from Ashfork	US 66	Both	3,253	8,735c	17.5	11.5	11.1	11.0	10.9	10.9	10.9	10.8
19 mi from Globe	US 70	Both	1,768	4,220c	16.1	9.4	9.0	8.9	8.9	8.8	8.7	8.7

Location	Route	Direction										
Colorado												
1 mi from Sedalia	SH 67	Both	611	2,788c	42.4	38.8	34.9	32.6	31.8	30.9	26.5	20.8
1 mi W of Ault	SH 14	Both	1,465	2,386c	14.8	13.4	12.8	12.4	12.2	11.9	10.9	10.0
Idaho												
8.2 mi SE of jct. with SH 21 in Boise	US 30	Both	5,455	8,807	14.4	12.6	12.4	12.1	11.8	11.6	10.9	10.0
8.7 mi SE of Center St. in Pocatello	US 91, US 191, US 30	Both	4,537	7,300c	13.7	12.3	11.6	11.2	10.2	10.8	10.0	9.3
3.0 mi E of jct. with US 410 in Lewiston	US 95	Both	3,802	5,647c	14.6	13.0	12.3	11.8	11.4	11.2	10.5	9.8
Montana												
10 mi E of Missoula	I 90	Both	2,651	5,306c	15.3	14.7	14.4	14.1	14.0	13.9	13.3	—
1 mi E of East Helena	US 12 & SH 287	Both	1,958	4,497c	18.5	17.1	16.3	15.8	15.5	15.2	13.7	—
1 mi W of Superior	I 90	Both	1,847	4,389	19.6	17.3	16.6	16.4	16.1	15.8	15.1	—
8 mi N of West Yellowstone	US 191	Both	1,041	3,514c	31.2	28.3	26.9	26.2	25.8	25.5	23.7	—
2 mi from Havre	US 87 & US 2	Both	2,326	3,576	15.7	11.1	10.7	10.4	10.4	10.3	9.7	—
Nevada												
15 mi from Las Vegas	US 95	Both	4,137	7,527c	25.7	18.5	17.6	17.0	16.6	16.3	15.3	13.7
0.5 mi W of W city limits of Tonopah	US 6 & US 95	Both	1,274	2,164c	17.7	12.2	11.2	10.9	10.7	10.4	10.0	9.3
New Mexico												
W of Tucumcari	US 66	Both	4,425	10,841c	17.2	15.1	14.6	13.8	13.2	12.8	12.1	11.3
3.5 mi S of Taos	US 64	Both	3,825	7,908c	16.2	14.1	13.9	13.4	13.2	12.9	11.9	11.0
16.5 mi from Albuquerque	NM 10	Both	1,784	5,395c	37.1	29.6	28.1	27.6	26.2	25.3	22.3	18.6
9.8 mi S of Roswell	US 285	Both	4,198	6,119c	13.3	11.9	11.6	11.4	11.2	11.1	10.5	9.8
3.3 mi W of Deming	US 70 & US 80	Both	3,649	9,830c	17.6	14.6	13.4	12.7	12.2	11.9	10.9	—
6.5 mi from Los Lunas	US 85	Both	3,198	4,981c	14.6	13.1	12.6	12.0	11.8	11.5	10.9	—
Utah												
1 mi from Sigurd	US 89	Both	2,423	4,949c	22.3	14.2	13.2	12.6	12.0	11.8	—	—
1 mi from Grantsville	US 40	Both	2,458	5,291c	14.6	12.4	12.1	11.9	11.7	11.5	—	—
Wyoming												
8.6 mi from Casper	US 20, US 26, US 80	Both	2,548	4,721c	14.0	12.4	11.7	11.5	11.5	11.3	—	—
11.4 mi from Cody	US 14	Both	846	4,008c	37.9	33.2	31.1	30.5	29.1	28.3	—	—
5.5 mi E of Gillette E corporate limit	US 14 & US 16	Both	1,659	3,480	17.7	16.5	15.8	15.4	15.3	14.9	—	—
7 mi N of Kemmerer N city limit	US 189	Both	446	941c	20.6	17.7	16.4	15.5	15.2	14.8	—	—
8 mi N of Carpenter	FAS 1103	Both	248	442	33.5	21.8	19.0	17.3	16.5	15.7	—	—

LOCATION OF COUNT STATION	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
			AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
Pacific												
Oregon												
E of Gresham	US 26	Both	10,069	15,198c	13.3	12.4	12.0	11.7	11.4	11.2	10.5	—
2 mi N of Newberg	US 99	Both	7,070	12,436c	15.7	14.5	13.6	13.1	12.8	12.5	11.2	—
Washington												
2.8 mi from Cleelum	US 10	Both	6,607	14,958c	23.0	18.0	17.0	16.0	15.6	15.2	13.4	11.9
6.0 mi N of Cheney	PSH 11	Both	6,193	9,827	12.4	10.7	10.3	10.2	10.0	9.8	9.3	8.9
Jct. PSH 9 & PSH 9 at Discovery Bay, NW leg	PSH 9	Both	2,457	5,854c	24.7	20.2	18.4	17.6	16.9	16.5	14.9	13.0
Jct. SSH 8A & SSH 1-U, 3.1 mi E of Vancouver		Both	3,176	4,988	11.9	10.9	10.6	10.4	10.4	10.2	9.9	9.2
Jct. PSH 3 (US 410) & PSH 3, Wallula jct. E leg, 16.4 mi from Walla Walla	PSH 3	Both	2,275	3,537c	13.8	12.4	12.2	11.7	11.5	11.4	10.6	9.8
1.2 mi N of Freeman	SSH3–H	Both	1,627	2,842c	17.3	14.8	14.1	13.6	13.4	13.0	11.9	10.6
2.0 mi N of jct. SSH 11-G & SSH T.C. near Othello	SSH11–G	Both	1,893	3,005	16.6	11.3	10.7	10.3	10.1	9.9	9.2	8.8
1.5 mi E of Tenino	SSH5–H	Both	794	2,380c	20.5	16.4	15.5	15.0	14.2	14.0	12.8	11.5
Alaska												
Anchorage-Seward Hwy., Anchorage	FAP 31	Both	8,960	13,287c	15.7	10.7	10.4	10.3	10.2	10.1	9.6	8.9
2.9 mi S of Eagle River	FAP 42	Both	3,089	7,027c	20.0	16.9	16.4	15.5	15.0	14.8	13.5	12.5
0.1 mi S of Potter Creek	FAP 31	Both	1,090	4,364c	48.3	38.3	34.2	32.1	31.1	29.8	25.5	17.9
1.1 mi N of Juneau NW city limit	FAP 95	Both	1,330	3,431c	26.2	20.0	19.2	18.2	17.4	16.8	—	—
1.0 mi N of Ketchikan W city limit	FAS 920	Both	1,507	2,179	16.9	14.4	13.9	13.6	13.5	13.3	—	—

For explanation of notes see Table A.1.

TABLE A.5—VARIATIONS IN TRAFFIC FLOW ON URBAN FREEWAYS

LOCATION OF COUNT STATION	NO. OF LANES	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
				AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
New England													
Maine													
0.7 mi W of US 1A, Bangor	4	I 395	EB			19.6	13.3	12.9	12.1	11.7	11.4	—	—
			WB			21.4	11.5	10.1	9.8	9.6	9.5	—	—
			Both	7,120	10,534								
1.7 mi from Augusta	4	I 95	NB			32.4	22.5	20.8	18.9	17.3	16.5	—	—
			SB			32.1	23.2	20.5	19.2	17.7	17.3	—	—
			Both	2,167	6,013[c]								
Massachusetts													
2.3 mi W of Mattapoisett	4	US 6	Both	15,988	25,509[c]	16.8	11.4	11.0	10.9	10.7	10.6	10.2	9.7
New Hampshire													
0.5 mi from Concord	4	I 93	Both	11,804	20,737	20.4	15.9	15.3	14.8	14.3	13.9	—	—
0.6 mi from Concord	4	I 93	Both	11,554	21,167[c]	21.4	16.6	15.6	14.9	14.5	14.1	12.5	—
0.5 mi S. Jct. US 3	4	I 93	Both	4,035	15,925[c]	24.7	20.2	19.1	18.2	17.4	16.5	14.2	—
2.0 mi from Manchester	4	I 193	Both	6,352	9,622	15.3	14.3	13.7	13.4	13.1	12.8	12.1	11.2
Rhode Island													
Pawtucket River Bridge	6	I 95	EB			11.2	8.8	—	—	—	—	—	—
			WB			11.1	10.3	9.8	8.6	8.2	—	—	—
			Both	19,216	24,293	10.1	9.7	9.2	8.7	8.3	8.2	—	—
Middle Atlantic													
New York													
Long Island Expwy. at 82nd St.	6	I 495	EB			9.4	8.4	8.2	8.1	8.0	7.9	7.6	7.1
			WB			9.4	8.8	8.6	8.3	8.2	8.1	7.6	7.1
			Both	127,910	157,940								
15 mi from N. Y. City, Nassau Co.	6	NYS 495	Both	119,300[b]	—	8.4	8.3	8.3	8.3	8.2	8.2	8.1	7.9
Cross Island Pkwy. at 114th Ave., New York City	6		NB			11.9	10.6	10.3	10.1	10.0	10.0	9.6	8.8
			SB			14.4	13.4	12.8	12.6	12.1	11.8	10.8	9.9
			Both	66,610	92,000[c]								
New England Thrwy., N. Y. City	6		NB			12.4	10.7	10.0	9.6	9.5	9.3	9.0	8.4
			SB			12.6	11.4	10.9	10.8	10.6	10.5	10.0	9.2
			Both	47,420	65,970[c]								

LOCATION OF COUNT STATION	NO. OF LANES	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
				AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
Henry Hudson Pkwy., N. Y. City	6		NB			14.3	13.3	12.7	12.2	11.8	11.5	10.5	9.8
			SB			15.1	13.7	13.3	13.1	12.9	12.8	11.8	10.9
			Both	37,310	38,700								
Rockaway Blvd. at Queens; N. Y. City	6		NB	37,690	49,170	14.4	12.6	11.7	11.4	11.1	10.9	10.3	9.6
Pennsylvania													
Schuylkill Expwy. 2 mi from Philadelphia	6	I 805	EB			11.0	10.6	10.5	10.5	10.4	10.4	—	—
			WB			10.4	9.9	9.8	9.8	9.7	9.7	—	—
			Both	113,564	134,855	8.9	8.8	8.7	8.6	8.6	8.6	8.4	8.2
Allegheny Co., 4 mi from Pittsburgh	4	I 70	EB			12.4	11.9	11.7	11.5	11.3	11.2	—	—
			WB			11.6	11.4	11.3	11.2	11.1	11.1	—	—
			Both	55,896[b]	70,290								
Fort Pitt Tunnel, 1 mi from Pittsburgh	4	I 70	EB			13.1	12.8	12.7	12.7	12.6	12.6	—	—
			WB			15.0	14.4	14.1	14.0	13.9	13.8	—	—
			Both	47,489	58,012	10.9	10.7	10.6	10.5	10.5	10.4	10.1	9.9
W end of South (John Harris) Br., 3.5 mi from Harrisburg	4	I 83	NB			13.8	12.9	12.6	12.5	12.4	12.3	—	—
			SB			14.5	12.9	12.6	12.5	12.3	12.3	—	—
			Both	36,974	47,818	11.7	11.1	10.9	10.7	10.6	10.4	10.0	9.6
E. app. to Penrose Ave. Br., 5 mi from Philadelphia	6	Pa 291	EB			14.3	12.7	12.5	12.3	12.2	12.1	—	—
			WB			12.0	11.5	11.4	11.2	11.2	11.1	—	—
			Both	47,010	57,381	12.0	11.3	11.1	11.0	10.9	10.8	10.6	10.3
South Atlantic													
Maryland													
S of US 40W, 0.9 mi from Catonsville	4	I 695	NB			11.0	10.2	10.0	10.1	—	9.7	—	—
			SB			10.3	9.9	9.8	9.7	—	9.3	—	—
			Both	40,763	—	9.0	8.7	8.6	8.5	—	8.3	—	—
S of US 1, 0.8 mi from Fullerton	4	I 695	NB			14.8	14.2	13.9	13.7	—	13.4	—	—
			SB			15.7	14.9	14.7	14.6	—	14.4	—	—
			Both	27,446	—	11.2	10.7	10.6	10.5	—	10.4	—	—
West Virginia													
Fort Henry Br., 9th & Main St., 3.0 mi from Wheeling	4	US 40 & 250	EB			17.9	15.3	14.1	13.6	13.1	12.6	—	—
			WB			12.0	11.2	10.8	10.7	10.5	10.4	—	—
			Both	21,692	37,006[c]								

Location	Lanes	Route	Dir.	Vol.	Vol.								
Florida													
US 92, approx. 4.0 mi NE of St. Petersburg	4	SR 600	EB			15.2	13.1	12.4	12.2	12.0	11.6	10.5	—
			WB			23.0	15.2	13.9	13.5	13.3	12.9	11.4	—
			Both	9,415	—								
East North Central													
Illinois													
Congress St. Expwy., Chicago	8	I 90	EB			16.5	14.8	14.6	14.4	14.3	14.2	13.8	13.5
			WB			—	—	—	—	—	—	—	—
			Both	89,000[b]	118,680	10.9	10.2	10.1	9.9	—	—	—	—
Kingery Expwy. at Wentworth Ave., Lansing	4	I 80–90–294	EB			14.1	13.0	12.7	12.2	11.9	11.7	10.8	10.1
			WB			—	—	—	—	—	—	—	—
			Both	27,600[b]	44,570	10.7	10.2	9.9	9.7	—	—	—	—
Edens Expwy. 3.4 mi from Highland Park	6	FAP 199	NB			12.2	11.6	11.5	11.3	11.0	10.8	10.4	9.9
			SB			—	—	—	—	—	—	—	—
			Both	27,000[b]	35,800[c]	9.9	9.4	9.3	9.2	—	—	—	—
West North Central													
Kansas													
0.1 mi NE of Lenexa	4	I 35	Both	10,786	15,097[c]	17.2	11.5	11.0	10.8	10.7	10.6	10.2	—
East South Central													
Mississippi													
1.0 mi W of US 45, Meridian	4	US 11 & 80	EB			10.6	10.1	9.7	9.6	9.5	9.3	9.0	8.4
			WB			11.4	9.5	9.1	8.9	8.8	8.7	8.3	7.9
			Both	10,199	14,748	10.1	9.3	9.0	8.9	8.9	8.8	8.4	7.9
2.5 mi S of Jackson	4	US 51	NB			19.3	13.9	12.9	11.8	11.5	11.3	10.5	9.6
			SB			18.5	14.0	13.3	12.5	12.1	11.7	11.1	9.9
			Both	6,242	8,894[c]	12.8	11.0	10.6	10.4	10.2	10.0	9.4	8.7
Tennessee													
0.6 mi from Knoxville	4	I 75	NB			16.3	15.3	15.0	14.8	14.6	14.4	13.8	12.6
			SB			16.3	15.2	14.7	14.5	14.2	13.9	13.1	11.7
			Both	20,417	28,230								
4.6 mi from Nashville	4	Tenn 1	EB			12.7	11.8	11.6	11.3	11.1	11.0	10.5	10.0
			WB			12.4	11.3	11.1	10.9	10.8	10.7	10.4	9.8
			Both	24,129	33,615[c]								
Magnolia Expwy., Knoxville	4	I 40 & 75	EB			12.8	12.0	11.9	11.8	11.6	11.4	10.9	9.6
			WB			11.5	10.5	10.3	10.2	10.2	10.1	9.7	9.2
			Both	21,785	28,743								
West South Central													
Arkansas													
Miss. R. Br., 1.0 mi from Memphis	4	I 55	NB			15.7	13.1	10.9	10.3	10.0	9.8	9.3	8.3
			SB			16.6	14.8	13.6	13.2	12.6	12.5	10.4	8.8

TABLE A.5—VARIATIONS IN TRAFFIC FLOW ON URBAN FREEWAYS (CONT.)

LOCATION OF COUNT STATION	NO. OF LANES	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
				AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
Louisiana													
New Orleans Expwy., New Orleans	6	I 10	EB			21.5	19.1	18.4	18.1	17.7	17.3	15.9	14.1
			WB			22.2	18.4	17.8	17.7	17.5	17.3	16.7	15.2
			Both	38,542	50,693	21.4	12.4	12.0	11.8	11.7	11.6	11.2	10.8
Oklahoma													
N of jct. of I 40, Oklahoma City	6	I 35	Both	19,157	—	14.8	12.1	11.5	11.3	11.0	10.7	8.2	—
Texas													
2.0 mi from San Antonio	4	I 10	NB			14.2	13.9	13.7	13.6	13.6	13.5	13.2	12.4
			SB			15.5	14.7	14.5	14.3	14.2	14.1	13.8	12.3
			Both	52,000	65,261								
1.0 mi from San Antonio	4	I 35	NB			15.0	14.5	14.3	14.2	14.1	13.9	13.4	12.0
			SB			12.3	11.6	11.3	11.2	11.1	11.0	10.7	10.3
			Both	41,172	48,544								
Gulf Fwy. 6 mi from Houston	6		NB			12.4	12.1	11.9	11.7	11.6	11.3	10.3	9.1
			SB			12.9	12.7	12.6	12.5	12.4	12.3	11.8	10.4
			Both	70,180	85,995								
Central Expwy. at Ross Ave., Dallas	6	US 75	NB			14.8	13.7	13.5	13.2	13.0	12.9	12.1	11.2
			SB			14.8	13.5	13.1	13.0	12.9	12.8	12.4	11.9
			Both	60,406	78,210								
1.5 mi from Austin	4	I 35	NB			17.5	16.0	15.7	15.5	15.2	15.1	14.6	12.9
			SB			16.5	15.8	15.6	15.4	15.2	15.1	14.6	9.7
			Both	30,953	44,286								
1.5 mi from Ft. Worth	6	I 35W	NB			13.7	13.2	13.1	13.0	12.9	12.9	12.6	11.5
			SB			13.5	12.7	12.3	12.2	12.1	12.0	11.8	11.0
			Both	51,677	66,432								
6.7 mi from Houston	6	I 45	NB			14.4	13.5	13.1	13.1	12.9	12.8	11.8	10.0
			SB			15.0	12.6	12.3	12.1	11.9	11.8	11.1	10.3
			Both	44,920	62,798								
East Texas Fwy., 1.0 mi from Houston	6		NB			14.6	14.0	13.8	13.7	13.6	13.5	12.8	12.1
			SB			13.9	13.2	13.0	13.0	12.9	12.8	12.5	12.0
			Both	41,170	49,538								

Location	Lanes	Route	Dir										
Texas (Cont'd)													
1 mi from San Antonio	4	I 35	NB			13.9	13.0	12.7	12.5	12.4	12.4	12.0	11.1
			SB			14.3	13.8	13.4	13.3	13.1	13.1	12.5	11.6
			Both	29,954	41,499								
1 mi from Beaumont	4	I 10	EB			11.8	11.2	11.1	10.9	10.8	10.7	10.2	9.6
			WB			13.5	12.8	12.3	12.0	11.9	11.8	11.1	10.4
			Both	22,867	30,609								
2.0 mi from Beaumont	4	I 10	EB			13.9	12.0	11.7	11.5	11.4	11.2	10.8	10.3
			WB			12.3	11.6	11.4	11.2	11.1	11.0	10.5	9.7
			Both	18,992	24,729c								
5 mi from Fort Worth	6	I 820	NB			17.4	16.7	16.4	16.2	16.1	15.9	15.4	14.6
			SB			19.2	18.2	17.9	17.7	17.5	17.5	17.1	16.2
			Both	8,639	10,920								
Mountain													
Colorado													
3 mi from Denver	6	I 25	NB			16.0	14.7	14.4	14.0	13.8	13.5	13.2	12.6
			SB			13.1	12.4	12.2	12.0	11.7	11.6	11.3	10.9
			Both	56,400	75,151	13.4	13.0	12.6	12.4	12.2	12.0	11.6	11.3
Wyoming													
2.7 mi from Cheyenne	4	I 25,	NB			17.8	14.6	14.1	13.4	12.9	12.2	—	—
		US 85	SB			17.8	14.4	13.9	13.3	12.8	12.4	—	—
		US 87	Both	1,656	2,264								
Pacific													
Oregon													
Columbia R. Hwy., Portland	6	I 80N	EB			17.0	16.0	15.7	15.6	15.5	15.4	—	—
			WB			19.1	17.7	17.4	17.1	16.9	16.6	—	—
			Both	44,342	60,502	11.5	11.1	11.0	10.8	10.7	10.7	10.5	—
Pacific Hwy., 3 mi from Portland	6	I 5	NB			16.4	15.6	15.1	15.0	14.8	14.6	14.1	—
			SB			15.9	15.1	14.7	14.6	14.5	14.4	13.8	—
			Both	31,922	42,383	12.1	11.1	10.7	10.5	10.5	10.4	10.1	9.7
Eugene-Springfield Hwy., 2 mi from Eugene	4	I 105	Both	12,230	18,017	13.4	11.7	11.1	11.0	10.7	10.7	10.0	9.6

For explanation of notes see Table A.1.

TABLE A.6—VARIATIONS IN TRAFFIC FLOW ON URBAN EXPRESSWAYS

LOCATION OF COUNT STATION	NO. OF LANES	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
				AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
New England													
Rhode Island													
0.5 mi from East Providence	6	I 195	NB			.15.9	13.7	12.4	—	—	—	—	—
			SB			15.3	14.1	9.8	—	—	—	—	—
			Both	28,625	35,182	11.5	10.5	10.1	9.6	9.2	9.0	—	—
Middle Atlantic													
New York													
21 mi from New York City	6	NYS 27	Both	20,800[b]	—	11.6	10.4	9.8	9.5	9.4	9.1	8.5	7.8
South Atlantic													
Delaware													
1 mi from Newark	4	Del. 2	Both	14,755[b]	19,519[c]	11.9	11.2	11.1	10.7	10.6	10.5	—	—
East South Central													
Mississippi													
3 mi from Jackson	2	US 80	Both	12,103	15,330	13.3	9.5	9.1	9.0	8.9	8.7	8.3	7.7
Natchez Br. over Miss R. in Natchez	2	US 65 & US 84	Both	10,363	14,253[c]	10.7	9.5	9.2	9.0	8.9	8.7	8.3	7.9
3 mi from Gulfport	4	US 90	EB			9.6	8.6	8.3	8.2	8.2	8.1	7.9	7.6
			WB			14.2	10.3	10.0	9.8	9.7	9.6	9.3	8.9
			Both	18,703	25,658[c]	10.2	9.0	8.8	8.7	8.6	8.6	8.2	7.9
Tennessee													
East Parkway at Southern R. R	4		NB			10.7	9.8	9.6	9.6	9.4	9.4	9.1	8.2
Underpass in Memphis			SB			10.7	10.1	9.9	9.7	9.7	9.6	9.4	8.7
			Both	29,929	36,414								
West South Central													
Arkansas													
1 mi from Little Rock	4	SH 10	EB			17.5	16.5	16.1	15.6	15.4	15.2	—	—
			WB			20.7	17.7	17.3	17.1	16.9	16.8	—	—
			Both	16,000	18,900								
Oklahoma													
W of Classen Circle, Oklahoma City	4	I 440	Both	39,143	—	10.9	10.3	10.0	9.8	9.6	9.5	8.6	—
Between Lincoln Blvd. and Kelley Ave., Oklahoma City	4	I 440	Both	21,552	—	11.8	10.0	9.7	9.6	9.5	9.3	8.9	—

Texas													
0.5 mi from Lubbock	6	US 84	NW			12.9	,10.8	10.4	10.3	10.2	10.1	9.6	9.0
			SE			17.4	13.8	13.5	13.3	13.0	12.9	12.1	10.8
			Both	13,000	18,902c								
Mountain													
Arizona													
0.8 mi from Tempe	4	US 60	Both	22,777	31,647	11.3	9.5	9.3	9.2	9.2	9.2	9.1	9.1
4 mi from Tucson	4	I 10	NB			16.5	8.9	8.7	8.6	8.5	8.5	8.5	8.4
			SB			16.2	9.2	8.8	8.8	8.7	8.6	8.5	8.4
			Both	9,866	12,510c								
1 mi from Yuma	4	US 80	Both	6,890	9,996c	11.0	8.0	7.3	.7.2	7.2	7.2	7.1	7.1
Colorado													
Speer Blvd. between Franklin and													
Gilpin Sts., Denver	6		EB			17.3	15.3	14.7	14.4	14.1	13.7	12.3	8.8
			WB			12.8	11.5	11.2	11.0	10.9	10.7	9.9	7.7
			Both	24,779	34,449	12.6	11.9	11.7	11.5	11.4	11.3	10.9	10.4
Pacific													
Oregon													
1 mi from Eugene	4	US 99W	Both	20,100	28,836	11.9	10.7	10.4	10.1	—	9.7	—	—
Washington													
Jct. PSH 1 & North 175th in Seattle	4	US 99	SB	49,432	62,886	12.4	10.9	10.8	10.6	10.6	10.5	10.1	9.1
Jct. PSH 2 & 61st Ave. in Seattle	4		EB	27,240	25,680	13.0	12.5	12.3	12.1	11.9	11.8	11.3	10.7

For explanation of notes see Table A.1.

TABLE A.7—VARIATIONS IN TRAFFIC FLOW ON CITY STREETS WITH MORE THAN TWO LANES

LOCATION OF COUNT STATION	NO. OF LANES	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
				AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
New England													
Maine													
1.4 mi from Portland	4	Rt 100	NB			11.8	10.3	9.5	8.6	7.6	—	—	—
			SB			8.8	7.7	7.2	6.7	6.2	—	—	—
			Both	14,636	17,958	10.3	9.7	9.4	9.3	9.2	9.2	—	—
0.6 mi from Bangor	4	US 1A	NB			8.8	7.8	7.1	6.8	6.4	6.1	—	—
			SB			11.0	9.0	7.6	7.0	6.5	6.2	—	—
			Both	18,171	23,417[c]	9.7	9.0	8.9	8.7	8.7	8.6	—	—
New Hampshire													
Main St. in Concord	4	US 3 & US 202	Both	13,518	18,722	10.3	9.7	9.5	9.4	9.3	9.2	—	—
Elm St. in Manchester	4	US 3	Both	11,587	15,323[c]	10.5	9.8	9.1	8.8	8.7	8.6	8.3	—
Rhode Island													
2 mi from Cranston	4	RI 2–3	Both	31,581	35,297	9.2	8.6	8.5	8.2	7.9	7.8	—	—
Pawtuxet R. Bridge, Warwick	4	RI 117	Both	22,612	35,260	12.0	10.7	10.1	9.9	9.7	9.4	—	—
2 mi from Warwick	4	RI 2	Both	19,150	37,492	19.3	13.6	11.7	10.8	9.9	9.4	—	—
0.1 mi from Newport	4	RI 138	Both	16,926	20,904[c]	9.5	8.0	7.7	7.4	7.3	7.2	—	—
Vermont													
0.2 mi from Winooski	4	US 7	Both	15,863	23,494[c]	11.4	10.7	10.4	10.2	10.0	9.9	9.5	—
0.5 mi from Montpelier	4	US 2	Both	4,608	6,742	15.8	14.9	14.3	14.0	13.6	13.0	11.6	—
Middle Atlantic													
New York													
1.5 mi from Larchmont	4	US 1	Both	16,200[b]	—	16.4	11.7	10.3	10.1	9.7	9.4	8.4	7.6
Pennsylvania													
3 mi from S Williamsport	4	US 15	NB			14.8	10.0	9.1	8.9	8.7	8.6	—	—
			SB			14.2	11.1	10.8	10.7	10.6	10.4	—	—
			Both	17,813	23,767	11.0	9.7	9.5	9.3	9.2	9.1	—	—
South Atlantic													
Maryland													
1.8 mi S of Glen Burnie	4	Md 2	NB			13.5	12.6	12.2	11.9	—	11.4	—	—
			SB			13.1	12.0	11.7	11.6	—	11.3	—	—
			Both	26,918	—	10.0	9.7	9.4	9.2	—	8.8	—	—

Location	Lanes	Route	Dir.										
Florida													
1.0 mi N of Miami Beach	4	SR A1A	NB			13.0	11.7	11.3	11.0	10.9	10.8	10.2	—
			SB			14.4	11.5	10.9	10.5	10.2	10.1	9.4	—
			Both	22,915b	—								
Tallahassee	4	US 90	EB			13.5	9.8	9.4	9.2	9.1	9.0	8.6	—
			WB			12.6	11.1	10.7	10.4	10.1	10.0	9.6	—
			Both	17,535	—								
1 mi from Orlando	4	US 441	Both	20,700	—	9.9	9.5	9.1	9.0	8.8	8.7	8.4	—
S of Warren Br., Jacksonville	4	US 17	Both	12,245	—	11.6	11.0	10.7	10.7	10.6	10.5	10.3	—
Georgia													
3.1 mi from Atlanta	6	US 19	Both	27,830b	35,662	10.5	9.7	9.6	9.4	9.4	9.3	8.1	—
2.5 mi from Atlanta	4	US 78	Both	19,660b	30,533	12.3	10.7	10.0	9.7	9.4	9.2	8.8	—
0.3 mi from Atlanta	4	US 19	Both	20,711b	26,503	10.5	9.6	9.3	9.2	9.0	9.0	8.7	—
East North Central													
Ohio													
3.5 mi from Cincinnati	4	US 50	EB			15.7	15.1	14.9	14.7	14.5	14.4	—	—
			WB			14.1	13.3	13.2	13.0	12.9	12.8	—	—
			Both	29,132	35,429	10.4	10.2	10.0	9.8	9.8	9.7	9.3	8.7
6.7 mi from Cincinnati	4	US 42	NB			11.0	10.8	10.5	10.4	10.3	10.3	10.0	9.6
			SB			10.1	9.6	9.5	9.4	9.3	9.3	9.1	8.8
			Both	22,658	27,312	9.8	9.4	9.3	9.2	9.1	9.0	8.8	8.6
Keowee St. at bridge, over Miami R.	4		NB			13.8	12.0	12.1	11.7	11.4	11.3	10.7	10.0
2 mi from Dayton			SB			13.1	11.8	11.5	11.3	11.1	11.0	10.5	9.8
			Both	13,921	19,072	11.6	10.1	9.9	9.7	9.5	9.4	8.7	8.2
Torrence Pkwy. bet. Madison &	4		NB			15.3	14.7	14.6	14.4	14.2	14.2	—	—
Columbia in Cincinnati			SB			15.1	14.5	14.4	14.2	14.1	14.0	—	—
			Both	11,117	13,568	12.2	12.0	11.9	11.7	11.7	11.6	11.4	11.1
West North Central													
Kansas													
Topeka Blvd. N of Hampton St.,	6		Both	22,597	28,912	9.9	9.5	9.3	9.2	9.0	9.0	8.7	—
Shawnee County, Topeka													
10th St. E of Plass, Topeka	4		Both	10,642	12,799	11.4	10.9	10.6	10.5	10.4	10.3	9.9	—
South Dakota													
S of 1st Ave. in Aberdeen	4	US 281	Both	9,237b	13,777c	11.9	11.1	10.5	10.2	—	—	—	—
Pierre St. RR viaduct, Pierre	4	US 83	Both	7,483b	14,328	14.6	11.8	11.3	10.8	—	—	—	—
East South Central													
Alabama													
3.1 mi from Huntsville	4	US 231	Both	14,096	18,636c	12.2	11.7	11.5	11.3	11.1	11.0	10.4	9.8

LOCATION OF COUNT STATION	NO. OF LANES	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
				AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
Mississippi													
N. State St. in Jackson	4	US 51	Both	23,512	32,628	11.9	10.6	10.4	10.1	9.9	9.8	9.3	8.9
0.05 mi E of US 49, Hattiesburg	4	US 98	EB			11.9	9.9	9.5	9.4	9.3	9.1	8.8	8.3
			WB			16.4	12.0	11.7	11.5	11.2	11.1	10.5	9.8
			Both	10,281	14,332c	12.6	10.4	10.1	9.8	9.7	9.6	9.3	8.8
Tennessee													
Woodland St. Br. in Nashville	4	US 31	NB			15.7	13.8	13.6	13.5	13.4	13.3	12.8	12.1
			SB			14.1	13.3	13.1	13.0	12.9	12.8	12.2	9.9
			Both	24,402	32,187c								
Arkansas-Tennessee River Br., Memphis	4	US 61	NB			15.5	12.9	11.1	10.7	10.1	9.9	9.2	7.8
		US 63	SB			16.0	15.4	13.6	13.3	12.8	12.4	10.8	8.5
		US 64	Both	25,779	37,174c								
West South Central													
Louisiana													
Gentilly Road, New Orleans	6	US 90	EB			10.8	10.2	10.0	9.8	9.8	9.7	9.3	8.6
			Both	40,120	47,202c								
Scenic Hwy., Baton Rouge	4	US 61	EB			13.8	12.4	12.0	11.7	11.5	11.3	10.8	9.8
			WB			19.4	12.0	11.5	11.2	11.1	10.9	10.5	9.9
			Both			12.3	10.5	10.2	10.1	10.0	9.9	9.6	9.1
Baton Rouge Br., Baton Rouge	4	US 190	EB			15.0	13.4	12.9	12.5	12.2	12.0	—	—
			Both	20,169	29,826	14.1	10.8	10.2	10.0	9.7	9.6	—	—
Huey P. Long Br., New Orleans	4		EB			14.2	13.2	13.1	13.0	12.9	12.7	12.3	11.5
			WB			12.5	11.9	11.6	11.5	11.4	11.3	10.9	9.9
			Both	21,044	25,082	11.1	10.5	10.3	10.2	10.1	10.0	9.8	9.5
Oklahoma													
Classen Blvd. N of N. 32, Oklahoma City	4		Both	25,332	—	11.0	10.5	10.4	10.3	10.2	10.1	9.8	—
Robinson Ave., Oklahoma City	4	US 62	Both	15,877	—	12.4	11.2	11.0	10.8	10.6	10.5	9.8	—
North 23 St., Oklahoma City	4	US 62	Both	17,313	—	10.1	8.9	8.7	8.6	8.5	8.3	7.5	—
Pa. Ave., N of N. 56th St. in Oklahoma City	4		Both	11,502	—	14.6	13.4	12.8	12.5	12.3	11.9	11.3	8.0
Santa Fe Ave. N of N. 36th St. in Oklahoma City	4		Both	5,284	—	22.1	17.0	16.4	16.1	15.3	14.9	12.7	—

Mountain													
Arizona													
Central & Lewis Ave., Phoenix	6		Both	32,273[b]	40,530	10.1	7.7	7.6	7.6	7.6	7.6	7.6	7.5
Colorado													
1 mi from Commerce City	4	US 6–85	NB			18.0	16.7	15.4	14.7	14.1	13.7	12.5	11.6
			SB			17.6	14.7	13.0	12.8	12.5	12.4	12.0	10.7
			Both	26,825	39,744	12.4	11.1	10.4	10.2	10.0	9.9	9.4	8.8
Idaho													
State St., Boise	4.	SH 44	Both	14,266	20,480	14.5	11.3	10.9	10.7	10.6	10.3	—	—
Americana Blvd., E end Boise R. Br., Boise	4		Both	13,810	19,929	14.3	13.0	12.1	11.9	11.6	11.5	—	—
10th Ave., Caldwell	4	US 30	Both	6,387	9,881	10.4	9.6	9.4	9.2	9.1	9:0	—	—
Montana													
Montana Ave., Helena	4	US 91	NB			21.1	12.7	11.4	10.9	10.7	10.6	10.0	—
			SB			12.1	11.1	10.8	10.6	10.4	10.3	9.7	—
			Both	8,263	10,834	11.8	10.9	10.5	10.4	10.3	10.2	9.7	—
Nevada													
3 mi from Las Vegas	4	US 93	EB			12.8	11.3	10.8	10.7	10.4	10.2	9:6	8.9
			WB			14.0	11.0	10.6	10.0	9.7	9.5	9.0	8.5
			Both	16,318	22,093	11.1	10.1	9.9	9.7	9.4	9.3	8.9	8.3
Between Reno & Sparks	4	US 40	Both	17,384	24,270[c]	9.7	9.1	8.8	8.7	8.6	8.6	8.3	8.0
New Mexico													
4.1 mi N of Albuquerque	4	US 85	Both	10,890	—	11.1	10.4	10.2	10.0	9.9	9.9	9.6	9.2
Utah													
2 mi from Salt Lake City	4	US 40	Both	16,536	26,450	16.3	12.3	12.0	11.7	11.5	11.3	—	—
Pacific													
Washington													
Garland Ave. in Spokane	4	PSH 3	NB			12.8	11.5	11.1	10.8	10.6	10.5	10.0	9.5
			SB			10.5	8.9	8.6	8.3	8.3	8.1	7.9	7.6
			Both	21,811	28,630[c]	10.7	9.9	9.4	9.2	8.9	8.8	8.5	8.2
Alaska													
L St. bet. 13th & 15th Ave. in Anchorage	4		Both	11,850[b]	16,213	10.3	9.4	8.9	8.8	8.6	8.4	—	—
5th Ave. bet. D & C Sts. in Anchorage	4		Both	8,370[b]	11,198	12.8	10.8	10.3	10.1	10.0	9.8	—	—

For explanation of notes see Table A.1.

TABLE A.8—VARIATIONS IN TRAFFIC FLOW ON TWO-LANE CITY STREETS

LOCATION OF COUNT STATION	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
			AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
New England												
Connecticut												
2 mi from West Hartford	US 6	Both	16,000[b]	26,228[c]	13.5	10.2	9.9	9.7	9.6	9.5	9.2	8.6
Maine												
1.5 mi from Lewiston	RT 126	Both	10,274	14,715[c]	11.2	10.4	10.1	10.0	9.8	9.8	—	—
0.7 mi from Hollowell	US 201	Both	9,696	12,129	11.1	10.5	10.0	9.9	9.7	9.7	—	—
0.1 mi from Sanford	Rt 4A–109	Both	8,055	11,064	12.5	10.5	10.3	10.1	9.9	9.8	—	—
New Hampshire												
1.5 mi from Concord	NH 13	Both	3,094	—	22.6	13.8	12.9	12.7	12.4	12.2	11.5	—
Rhode Island												
0.5 mi from Bristol	RI 114	Both	11,517	14,808[c]	10.6	9.7	9.2	9.0	8.8	8.6	—	—
Vermont												
Barre	US 302	Both	13,427	18,415	11.1	9.5	9.1	8.9	8.6	8.6	8.1	—
1 mi from Rutland	US 4	Both	9,393	13,673	12.3	11.2	10.9	10.7	10.6	10.3	10.0	—
St. Johnsbury	US 2	Both	7,013	11,712[c]	12.2	11.4	11.0	10.6	10.3	10.2	9.7	—
Middle Atlantic												
New Jersey												
0.5 mi from Plainfield, E. 7th St.		Both	9,259[b]	—	10.7	10.6	10.4	10.2	10.1	10.0	9.7	9.5
S. Grove St., East Orange		Both	6,606[b]	—	11.1	10.6	10.4	10.1	9.8	9.7	9.6	9.3
New York												
Watertown	US 11	Both	10,200[b]	—	11.8	9.5	9.1	8.9	8.7	8.6	8.2	7.8
Hornell	RT 36	Both	10,200[b]	—	9.8	9.0	8.8	8.7	8.6	8.6	8.3	8.0
South Atlantic												
Maryland												
0.1 mi from Riviera Beach	MD 173	Both	10,273	—	12.1	11.5	11.1	10.9	—	10.6	—	—
0.7 mi from Cockeysville	MD 45	Both	8,607	—	12.2	11.5	11.4	11.2	—	11.1	—	—
North Carolina												
1 mi from Wilmington, on NE Cape Fear R. Br.		Both	12,850	20,085	15.7	12.2	11.5	11.2	11.1	11.0	10.3	—
0.6 mi NW of New Bern city limit	US 70	Both	7,970	11,328[c]	11.4	10.6	10.2	9.9	9.8	9.7	9.1	—
South Carolina												
Cooper R. Br., Charleston	US 17	Both	13,258	17,696[c]	11.0	10.0	9.8	9.7	9.6	9.5	9.2	8.7

West North Central												
Iowa												
E leg 17th St. & Mt. Vernon in Cedar Rapids		Both	11,215	14,331	11.2	10.5	10.3	10.1	10.0	9.9	—	—
E leg Harrison & E. Washington, Mt. Pleasant	US 34	Both	7,169	12,280	13.0	12.0	10.7	10.2	9.8	9.7	—	—
N leg 338 St. & Lake in Storm Lake		Both	4,328	5,390	11.9	10.7	10.1	9.8	9.7	9.6	—	—
W leg 11th St. & Main in Adel		Both	1,180	1,657	15.8	12.6	11.6	11.3	11.1	11.0	—	—
Kansas												
Curtis St. E of Topeka Blvd. in Topeka		Both	863	1,544	15.9	14.0	13.3	13.1	12.9	12.6	12.1	—
High St. S of 13th St., Topeka		Both	533	689[c]	15.2	13.7	13.1	12.6	12.2	12.0	11.3	—
North Dakota												
0.5 mi from Minot		Both	3,750	5,600	13.1	12.3	11.6	11.3	11.1	11.0	—	—
East South Central												
Alabama												
8.0 mi from Birmingham	US 11	Both	19,880[b]	24,582	9.5	9.2	9.0	8.9	8.9	8.8	8.6	8.1
Kentucky												
Midland Ave., in Lexington	US 60	Both	10,220	—	10.2	9.1	8.9	8.8	8.7	8.7	—	—
Tennessee												
Western Ave., between Webster & Leslie in Knoxville	SR 62	Both	12,333	15,083	9.4	9.1	8.9	8.8	8.6	8.6	8.3	8.0
West South Central												
Arkansas												
2 mi from Little Rock	US 70	Both	17,600	19,200[c]	10.7	9.6	8.8	8.4	8.3	8.2	—	—
W 7th St. between Broadway & Spring in Little Rock		Both	8,650	10,150	10.8	9.9	9.6	9.4	9.2	9.1	8.7	8.3
Oklahoma												
10th St. N of Pine in Enid		Both	5,638	—	15.8	14.0	13.0	12.1	11.5	11.3	10.7	9.6
Strothers St. between University & Jefferson Sts., Seminole		Both	5,601	—	15.8	12.6	11.7	11.4	11.1	10.9	10.3	9.3
D St. between 13th & 14th Sts. in Lawton		Both	5,654	—	11.8	10.7	10.5	10.3	10.2	10.1	9.8	8.6
800 W. Fondulac St., Muskogee		Both	3,583	—	14.5	11.8	11.4	11.3	11.1	11.0	10.4	9.0
11th St. bet. Delaware & Wyandotte in McAlester		Both	2,474	—	13.2	12.0	11.6	11.3	11.2	11.0	10.5	9.7
Cedar St. bet. 9th & 10th in Perry		Both	1,179	—	24.6	15.0	12.6	12.0	11.5	11.3	10.4	8.5
Kentucky Ave. N of No. 32nd St. in Oklahoma		Both	328	—	33.8	21.0	18.9	17.7	17.4	16.2	14.3	11.0
Mountain												
Idaho												
11th Ave. in Nampa	US 30	Both	9,282	12,551[c]	13.4	10.1	9.8	9.6	9.4	9.2	—	—
Latah St. bet. Morris Hill Rd. & Tulare Dr. in Boise		Both	6,498	8,913	14.4	11.5	10.7	10.6	10.5	10.4	—	—
Montana												
1 mi from Billings		Both	14,712	21,143[c]	14.1	12.4	11.9	11.5	11.2	11.0	10.6	—

TABLE A.8—VARIATIONS IN TRAFFIC FLOW ON TWO-LANE CITY STREETS (CONT.)

LOCATION OF COUNT STATION	ROUTE NUMBER	DIREC. OF TRAVEL	24-HOUR VOLUME		VOLUME IN SELECTED HIGHEST HOURS AS A PERCENTAGE OF ANNUAL AVERAGE 24-HOUR VOLUME (AADT)							
			AADT[a]	PEAK DAY	MAX.	10TH	20TH	30TH	40TH	50TH	100TH	200TH
Nevada												
Kietzke Lane bet. Reno-Sparks city limit		Both	13,303	17,234	12.0	11.2	10.5	10.4	9.8	9.7	9.0	8.0
250 ft W of Sutro St., Reno		Both	10,093	13,433	12.8	11.8	11.5	11.4	11.3	11.2	10.8	10.3
150 ft W of FAS 704, Sparks	FHS 727	Both	4,886	7,761	12.4	11.8	11.7	11.4	11.0	10.9	10.4	9.8
Utah												
9th E. and 4600 S., 2 mi from Murray		Both	14,950	19,766	13.2	12.4	12.2	11.8	11.6	11.4	—	—
1.3 mi from Cedar City	US 91	Both	3,859	7,512	15.9	10.9	10.5	10.2	10.0	9.7	—	—
Wyoming												
Riner Viaduct, Cheyenne	US 85 & US 87	Both	12,657	18,446[c]	11.0	10.4	9.9	9.7	9.5	9.3	—	—
Center St. underpass in Casper		Both	9,292	11,861	10.9	10.3	10.0	9.8	9.7	9.6	—	—
Laramie	US 30	Both	8,069	14,137[c]	12.3	11.4	10.6	10.1	10.0	9.8	—	—
Goose Greek Bridge, Sheridan	US 14 & US 87	Both	5,456	8,739	14.5	11.1	10.8	10.6	10.4	10.2	—	—
Pacific												
Oregon												
1 mi from Salem		Both	9,258	11,814	11.3	10.7	10.4	10.3	10.2	10.2	9.9	—
Washington												
4.0 mi S of Seattle	SSH 1K	Both	11,185[b]	13,885	9.3	9.1	9.0	9.0	8.9	8.9	8.6	8.3
29th Ave. & Grand Blvd., W leg, in Spokane		Both	4,295	6,024	13.9	11.9	11.3	10.8	10.7	10.6	10.1	9.6
Alaska												
C St. at Ship Creek in Anchorage		Both	9,540[b]	13,698	15.8	12.8	11.8	11.3	11.1	10.9	—	—
FAP 37 bet Ill. St. & FAP 61 in Fairbanks		Both	3,380	6,185[c]	17.7	14.4	13.5	12.8	12.1	11.9	10.9	10.0

For explanation of notes see Table A.1.

APPENDIX B
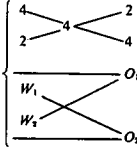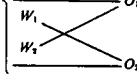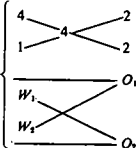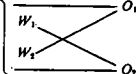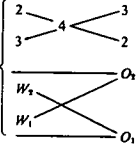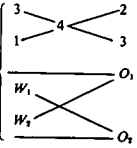
SELECTED OBSERVATIONS FROM 1963 BPR URBAN WEAVING AREA CAPACITY STUDY

TABLE B.1—SELECTED OBSERVATIONS FROM 1963 BPR URBAN WEAVING AREA CAPACITY STUDY

TYPE AND LOCATION	GEOMETRICS[a]	WEAVING MOVEMENT (vph) 6 MIN	18 MIN	1 HR	TOTAL MOVEMENT (vph) 6 MIN	18 MIN	1 HR	SPEED (mph) 6 MIN	18 MIN	1 HR	TIME 6 MIN	18 MIN	1 HR	V_{w_1} (vph) 6 MIN	18 MIN	1 HR	V_{w_2} (vph) 6 MIN	18 MIN	1 HR	$V_{w_1} + 3V_{w_2}$ (vph) 6 MIN	18 MIN	1 HR	V_{o_1} (vph) 6 MIN	18 MIN	1 HR	V_{o_2} (vph) 6 MIN	18 MIN	1 HR	LANE SCHEMATIC (DIRECTION OF FLOW)
Diamond aux. lane, Eastshore Fwy. N.B. at San Pablo off, Richmond, Calif.	2°-7° 917' 48'	1812	1545	1287	5616	5355	5040	33	32	38	5:10–5:16 p.m.	5:05–5:23 p.m.	4:25–5:25 p.m.			714			380			1854		96				3892	3>4<3 / 1<1
Cloverleaf, Eastshore Fwy. S.B. and University Ave., Berkeley, Calif.	18°-33° 425' 24'	1440	1364	1271	1460	1370	1276	34			5:00–5:06 p.m.	4:30–4:48 p.m.	4:24–5:24 p.m.	760	797	734	680	567	537	2800	2498	2345	0	3	1	20	3	4	1>2<1
Cloverleaf, Eastshore Fwy. S.B. and University Ave., Berkeley, Calif.	18°-33° 425' 24'	1780	1654	1550	1780	1657	1561	32	31	32	7:30–7:36 a.m.	7:24–7:42 a.m.	7:24–8:24 a.m.	1090	1087	928	690	567	622	3160	2788	2794	0	0	5	0	3	6	1>2<1
Cloverleaf, Bayshore Fwy. at Whipple Ave., Redwood City, Calif.	17°-29° 445' 48'	1960	1907	1666	5510	5443	5038	31	31	35	7:30–7:36 a.m.	7:18–7:36 a.m.	7:00–8:00 a.m.	980	1020	888	980	887	778	3920	3681	3222	0	0	0	3550	3536	3372	3>4<3 / 1<1
Cloverleaf, Bayshore Fwy. at Whipple Ave., Redwood City, Calif.	17°-29° 445' 48'	2280	2123	1775	5910	5503	4932	29	38	40	7:18–7:24 a.m.	7:18–7:36 a.m.	7:00–8:00 a.m.	1370	1246	1003	910	877	772	4100	3877	3319	0	0	0	3630	3380	3157	3>4<3 / 1<1
Cloverleaf, Bayshore Fwy. N.B. at San Bruno Ave., San Bruno, Calif.	23°-42° 449' 62'	1490	1563	1527	6570	6370	6095	41	37	33	6:54–7:00 a.m.	6:48–7:06 a.m.	6:42–7:42 a.m.	930	1033	811	560	530	716	2610	2623	2959	0	0	0	5080	4807	4568	4>5<4 / 1<1
Collector-distributor road for cloverleaf inner loops, 19th Ave. and Bayshore Fwy. N.B., San Mateo, Calif.	19°-31° 503' 24'	1400	1317	1048	1400	1317	1048	32	32	33	7:36–7:42 a.m.	7:36–7:54 a.m.	7:12–8:12 a.m.	820	750	501	580	561	461	2560	2451	1970	—	—	—	—	—	—	1>2<1
Diamond auxiliary lane, Hollywood Fwy. W.B. at Vermont on-ramp and Melrose off-ramp, Los Angeles, Calif.	1°-13° 1054' 62'	1750	1733	1510	6980	6963	6623	27	29	32	4:30–4:36 p.m.	4:24–4:42 p.m.	3:42–4:42 p.m.	1210	1120	956	540	613	554	2830	2959	2618	—	—	—	—	—	—	4>5<4 / 1<1
Cloverleaf, Edens Expy. S.B. and Dempster St., inner loop, Skokie, Ill.	33°-23° 710' 52'	1400	1240	1119	5700	5341	5010	39	42	44	8:06–8:12 a.m.	8:06–8:24 a.m.	7:36–8:36 a.m.	740	647	626	660	593	493	2720	2426	2105	0	0	0	4300	4101	3891	3>4<3 / 1<1
Cloverleaf, Edens Expy. S.B. and Touhy Ave., inner loop, Skokie, Ill.	31°-22° 726' 52'	1270	1110	950	4510	4029	3439	48	50	51	4:30–4:36 p.m.	4:18–4:36 p.m.	3:36–4:36 p.m.	960	780	640	310	330	310	1890	1770	1570	10	3	6	3230	2916	2483	3>4<3 / 1<1
Collector-distributor rd., Calif. on-ramp to Western off-ramp, Eisenhower Expy. E.B., Chicago, Ill.	8°-10° 868' 26'	1520	1447	1151	1520	1447	1190	40	38	37	7:06–7:12 a.m.	7:06–7:24 a.m.	7:06–8:06 a.m.	900	850	737	620	597	414	2760	2641	2211	0	0	0	0	0	39	1>2<1
Cloverleaf, Edens Expy. S.B. and Peterson Ave., inner loops, Chicago, Ill.	22°-27° 625' 52'	570	503	585	5350	5126	4808	34	32	34	4:36–4:42 p.m.	4:30–4:48 p.m.	4:24–5:24 p.m.	320	280	311	250	223	274	1070	949	1133	4780	4623	4223	0	0	0	3>4<3 / 1<1
On-ramp-off-ramp weave, Dan Ryan Expy. S.B. and ramp from Eisenhower Expy. E.B. and W.B., Taylor St., Chicago, Ill.	2°-7° 659' 64'	2980	2760	2492	6920	6634	5976	34	35	34	3:48–3:54 p.m.	3:36–3:54 p.m.	3:12–4:12 p.m.	2610	2467	2168	370	293	324	3720	3346	3140	20	91	78	3920	3783	3406	4>5<4 / 2<1

Location	Angle/Length																										
On-ramp-off-ramp weave, Dan Ryan Expy. S.B. and ramp from Eisenhower Expy. E.B. and W.B., Taylor St., Chicago, Ill.	2°-7° 659' 64'	2460	2474	2400	5800	5500	5401	43	39	37	7:06-7:12 a.m.	7:00-7:18 a.m.	7:00-8:00 a.m.	2040	1950	1891	420	524	509	3300	3522	3418	90	83	100	3250 2943 2911
Collector-distributor rd., Eisenhower Expy., Western to California Ave., Chicago, Ill.	8°-8° 844' 26'	1400	1320	1234	1430	1326	1240	38	38	40	4:12-4:18 p.m.	3:18-3:36 p.m.	3:18-4:18 p.m.	980	860	860	420	460	374	2240	2240	1982	20	3	4	10 3 2
Auxiliary lane, Eisenhower Expy., Racine Ave. to Ashland Blvd., Chicago, Ill.	6°-9° 1032' 72'	1570	1386	1355	6800	6807	6458	45	47	47	4:24-4:30 p.m.	4:12-4:30 p.m.	3:54-4:54 p.m.	1150	1003	956	420	383	399	2410	2152	2153	20	27	20	5210 5394 5083
Local lanes, auxiliary lane, Dan Ryan Expressway S.B., 51st St. to Garfield, Chicago, Ill.	1°-5° 894' 38'	1760	1620	1434	3480	3420	3237	36	37	37	4:36-4:42 p.m.	4:36-4:54 p.m.	4:18-5:18 p.m.	960	900	814	800	720	620	3360	3060	2674	1720	1800	1800	0 0 3
Local lanes, collector-distributor rd., Dan Ryan Expy. N.B., 55th St. to 51st St., Chicago, Ill.	5°-6° 890' 38'	1210	1214	1136	3400	3314	3134	36	37	39	7:12-7:18 a.m.	7:00-7:18 a.m.	7:00-8:00 a.m.	610	540	531	600	674	605	2410	2294	2198	2190	2100	1994	0 0 4
Local lanes, Dan Ryan Expy. S.B. Pershing to 43rd St., Chicago, Ill.	1°-7° 822' 50'	1320	1284	1111	5510	5190	4681	40	37	35	4:36-4:42 p.m.	4:30-4:48 p.m.	4:00-5:00 p.m.													
Local lanes, Dan Ryan Expy. S.B., 71st St. to 75th St., Chicago, Ill.	9°-14° 521' 62'	1260	1240	1256	7840	7607	7383	35	35	32	4:36-4:42 p.m.	4:30-4:42 p.m.	4:24-5:24 p.m.	890	817	848	370	423	408	2000	2086	2072	6580	6364	6122	0 3 5
Local lanes, Dan Ryan Expy. N.B., 63rd St. to 59th St., Chicago, Ill.	4°-5° 621' 50'	1460	1129	1111	4760	4217	3847	37	37	40	7:42-7:48 a.m.	7:30-7:48 a.m.	7:00-8:00 a.m.	990	823	836	470	306	275	2400	1741	1661	0	7	4	3300 3081 2732
Cloverleaf, South Conduit E.B. and inner loops from Van Wyck Expy., Long Island, New York, N.Y.	28°-48° 566' 42'	2710	2974	2890	5370	5130	4908	14[b] 16[c] 17[d]	17[b] 20[c] 23[d]	18[b] 21[c] 25[d]	5:00-5:06 p.m.	4:30-4:48 p.m.	4:06-5:06 p.m.	1700	1807	1831	1010	1167	1059	4730	5308	5008	0	0	3	2660 2157 2015
		—	—	2856	—	—	4868	— — —	— — —	25[b] 27[c] 31[d]	—	—	4:30-5:30 p.m.	—	—	1987	—	—	869	—	—	4594	—	—	1	— — 2031
		2490	2540	2365	3860	3817	3418	26[b] 29[c] 32[d]	27[b] 30[c] 34[d]	28[b] 30[c] 33[d]	7:30-7:36 a.m.	7:24-7:42 a.m.	6:54-7:54 a.m.	1470	1510	1253	1020	1030	1112	4530	4600	4589	0	0	4	1370 1277 1049
Service rd., simple weave, Van Wyck Expy. S.B. to North Conduit to Southern State Pkwy., Long Island, New York, N.Y.	24°-21° 625' 20'	2230	2044	1847	2910	2674	2364	31[b] 32[c] 34[d]	31[b] 32[c] 33[d]	33[b] 33[c] 33[d]	5:12-5:18 p.m.	5:00-5:18 p.m.	4:30-5:30 p.m.	1250	1100	927	980	944	920	4190	3932	3687	350	323	233	330 307 284
		3040	2947	2637	3570	3451	3123	29[b] 29[c] 29[d]	29[b] 29[c] 29[d]	31[b] 31[c] 31[d]	7:30-7:36 a.m.	7:18-7:36 a.m.	6:54-7:54 a.m.	2460	2283	2032	580	664	605	4200	4275	3847	140	133	108	390 371 378

TYPE AND LOCATION	GEO-METRICS[a]	WEAVING MOVEMENT (vph)			TOTAL MOVEMENT (vph)			SPEED (mph)			TIME			V_{w_1} (vph)			V_{w_2} (vph)			$V_{w_1} + 3V_{w_2}$ (vph)			V_{o_1} (vph)			V_{o_2} (vph)			LANE SCHEMATIC (DIRECTION OF FLOW)
		6 MIN	18 MIN	1 HR	6 MIN	18 MIN	1 HR	6 MIN	18 MIN	1 HR	6 MIN	18 MIN	1 HR	6 MIN	18 MIN	1 HR	6 MIN	18 MIN	1 HR	6 MIN	18 MIN	1 HR	6 MIN	18 MIN	1 HR	6 MIN	18 MIN	1 HR	
Major weave, outer connection S.E. quadrant, Van Wyck Expy. to Southern State Pkwy., Long Island, New York, N. Y.	17°-12° 980' 40'	3400	3319	3403	4800	4524	4467	29[b] 30[e] 32[d]	29[b] 29[e] 29[d]	25[b] 25[e] 25[d]	4:06– 4:12 p.m.	4:00– 4:18 p.m.	4:12– 5:12 p.m.	2936	2787	2953	464	532	450	4328	4383	4303	866	798	640	534	407	424	
		3550	3610	3436	5550	4920	4574	31[b] 29[e] 27[d]	31[b] 30[e] 28[d]	29[b] 29[e] 29[d]	5:06– 5:12 p.m.	4:54– 5:12 p.m.	4:18– 5:18 p.m.	2900	3123	3012	650	487	424	4850	4584	4284	990	733	665	910	577	473	
Cloverleaf, Northern State Pkwy. E.B. and entrance from Guinea Woods Rd. to Meadowbrook State Pkwy., Long Island, New York, N. Y.	31°-19° 564' 50'	2043	1906	2109	5080	4590	5039	31[b] 31[e] 31[d]	32[b] 35[e] 37[d]	24[b] 24[e] 23[d]	5:24– 5:30 p.m.	5:30– 5:48 p.m.	4:30– 5:30 p.m.	1688	1590	1691	355	316	418	2753	2538	2945	2645	2438	2566	392	246	364	
		2380	2330	2235	5480	5354	5048	22[b] 19[e] 14[d]	22[b] 20[e] 21[d]	18[b] 17[e] 16[d]	5:36– 5:42 p.m.	5:30– 5:48 p.m.	4:54– 5:54 p.m.	204	2033	1862	340	297	373	3060	2924	2981	2720	2707	2430	380	317	383	
		1920	1796	1691	4210	3837	3591	37[b] 39[e] 40[d]	38[b] 39[e] 40[d]	37[b] 39[e] 40[d]	7:36– 7:42 a.m.	7:30– 7:48 a.m.	7:24– 8:24 a.m.	1480	1369	1354	440	427	337	2800	2650	2365	1910	1757	1489	380	284	411	
Major weave, Van Wyck Expy. and ramps from Main St. and Queens Blvd. to Hillside Ave., Long Island, New York, N. Y.	6°-15° 497' 54'	—	2503	2451	—	4747	4622	— — —	28[b] 31[e] 34[d]	28[b] 32[e] 35[d]	— — —	3:30– 3:48 p.m.	3:24– 4:24 p.m.	—	2186	2180	—	317	271	—	3137	2993	—	30	23	—	2179	2148	
Major weave, Southern State Pkwy. and Bayshore Rd. ramp to Sagtikos Pkwy. and Heckscher St. Pkwy., Long Island, New York, N. Y.	5°-8° 1583' 50'	2090	1953	1828	3010	2786	2556	46[b] 45[e] 44[d]	48[b] 47[e] 46[d]	47[b] 47[e] 47[d]	4:48– 4:54 p.m.	4:48– 5:06 p.m.	4:30– 5:30 p.m.	1990	1740	1650	100	213	178	2290	2379	2184	300	300	197	620	533	531	

[a] Angle of convergence—angle of divergence; weave distance, in feet; weaving area width, in feet.   [b] Weaving speed.   [e] Average speed.   [d] Through speed.

APPENDIX C

SUMMARY OF STATISTICAL DATA FOR EQUATIONS OF FIGURES 8.2-8.19

This appendix summarizes, in Tables C-1 through C-4, the statistical data associated with the equations presented in Figures 8.2 through 8.19.

In the main tables the mean and standard deviations are given for each equation variable, as obtained from the data used to derive the equation by multiple linear regression. Next, the "range of use" of variables is given; this specifies the range within which the variable should be kept for an accurate use of the equation. If a range is given for a variable not included in the equation itself, the components preferably should be within the range given, inasmuch as conditions outside the given range could result in a different equation. However, there may well be situations which require extrapolation outside the variable range. Also given as statistical background are each of the net regression coefficients and their associated standard errors, as well as the coefficient of determination and the level of significance of each of the independent variables.

Other supplementary information is presented to the right of the main tables. First, the R^2 value, known as the coefficient of multiple determination, is shown for each equation. An R^2 of 0.90, for instance, indicates that the equation accounts for 90 percent of the variability of the dependent variable. Some factors not included in the equations which may account for some of the unexplained variance are such hard-to-measure items as trip lengths, signing effectiveness, and ramp action upstream or downstream of the ramp under consideration, but not adjacent to the ramp.

Other statistics given are the standard error of the dependent variable, the number of different locations used to provide data for the development of the equation, and the number of observations used in the analysis. Observations were not used if congested operation, usually consisting of stop-and-go accordion action, was present during the period counted.

TABLE C.1—STATISTICAL BACKGROUND OF EQUATIONS FOR LANE 1 VOLUME, 4-LANE FREEWAYS

FIG. NO.[a]	SITUATION DESCRIPTION, EQUATION, AND EQUATION VARIABLES[b]	REGRESSION STATISTICS										
		MEAN	STD. DEVI-ATION	RANGE OF USE	NET REGRESSION COEFFICIENT		PARTIAL DETERMI-NATION COEFF.	LEVEL OF SIGNIFI-CANCE	R^2	STD. ERROR OF V_1	NO. OF LOCA-TIONS	NO. OF OBSER-VATIONS
					VALUE	STD. ERROR						
8.2	*Simple on-ramp*											
	$V_1 = 136 + 0.345V_f - 0.115V_r$								0.83		20	431
	V_1, volume in lane 1 (vph)	736	262							113		
	V_f, freeway volume (vph)	1925	683	400–3400	+0.345	0.008	0.82	0.01				
	V_r, on-ramp volume (vph)	513	118	50–1400	−0.115	0.017	0.09	0.01				
8.3	*Off-ramp (no nearby adj. ramps)*											
	$V_1 = 165 + 0.345V_t + 0.520V_r$								0.90		19	518
	V_1, volume in lane 1 (vph)	1022	410							131		
	V_t, freeway volume (vph)	1828	828	400–4200	+0.345	0.008	0.79	0.01				
	V_r, off-ramp volume (vph)	434	307	50–1500	+0.520	0.021	0.73	0.01				
8.4	*Off-ramp with adj. upstrm. on-ramp*											
	$V_1 = 202 + 0.362V_t + 0.496V_r$											
	$\quad - 0.069D_u + 0.096V_u$								0.97		5	124
	V_1 = volume in lane 1 (vph)	1223	466							83		
	V_t = freeway volume (vph)	2440	963	700–4200	+0.362	0.013	0.87	0.01				
	V_r = off-ramp volume (vph)	497	325	50–1600	+0.496	0.038	0.59	0.01				
	D_u = dist. to adj. upstrm. on-ramp (ft)	1907	1034	700–3200	−0.069	0.011	0.25	0.01				
	V_u = vol. of adj. upstrm. on-ramp (vph)	236	211	50–900	+0.096	0.055	0.02	0.10				
8.5	*Cloverleaf on-ramp, no aux. lane*											
	(a) *(Ramp <600 vph)*											
	$V_1 = 166 + 0.280V_f$								0.42		6	75
	V_1 = volume in lane 1 (vph)	517	132							102		
	V_f = freeway volume (vph)	1254	305	600–2000	+0.280	0.039	0.42	0.01				
	V_r = on-ramp volume (vph)	359	168	0–599								
	(b) *(Ramp 600–1200 vph)*											
	$V_1 = 128 + 0.482V_f - 0.301V_r$								0.82		4	51
	V_1 = volume in lane 1 (vph)	460	254							111		

	V_f = freeway volume (vph)	1190	409	600–2000	+0.482	0.047	0.69	0.01				
	V_r = on-ramp volume (vph)	805	166	600–1200	−0.301	0.115	0.13	0.01				
8.6	*Cloverleaf on-ramp with aux. lane*								0.82		7	151
	$V_1 = 195 + 0.273V_f - 0.146V_r + 0.723V_d$									96		
	V_1 = volume in lane 1 (vph)	676	222									
	V_f = freeway volume (vph)	1468	672	600–3600	+0.273	0.012	0.79	0.01				
	V_r = on-ramp volume (vph)	479	379	100–1500	−0.146	0.023	0.22	0.01				
	D_d = dist. to adj. dnstrm. off-ramp (ft)			400–750								
	V_d = vol. of adj. dnstrm. off-ramp (vph)	208	100	50–500	+0.723	0.087	0.32	0.01				
8.7	*On-ramp with aux. lane to adj. off-ramp*								0.89		4	94
	$V_1 = 281 + 0.400V_f - 0.225D_d + 0.394V_d$									116		
	V_1 = volume in lane 1 (vph)	1079	336									
	V_f = freeway volume (vph)	2195	488	120–3200	+0.400	0.039	0.54	0.01				
	D_d = dist. to adj. dnstrm. off-ramp (ft)	1246	325	800–1700	−0.225	0.048	0.20	0.01				
	V_d = vol. of adj. dnstrm. off-ramp (vph)	506	253	50–1000	+0.394	0.089	0.18	0.01				
8.8	*Second of successive on-ramps*								0.93		7	148
	$V_1 = 123 + 0.376V_f - 0.142V_r$									76		
	V_1 = volume in lane 1 (vph)	829	285									
	V_f = freeway volume (vph)	2075	666	800–3600	+0.376	0.010	0.91	0.01				
	V_r = on-ramp volume (vph)	527	415	100–1500	−0.142	0.016	0.35	0.01				
	D_u = dist. to adj. upstrm. on-ramp (ft)	1002	497	400–2000								
	V_u = vol. of adj. upstrm. on-ramp (vph)	296	217	100–1000								

[a] Corresponding nomograph in Chapter Eight, giving situation sketch, equation, conditions for use, limiting factors, and steps in use of equation and nomograph.
[b] All variables independent except V_1.

TABLE C.2—STATISTICAL BACKGROUND OF EQUATIONS FOR LANE 1 VOLUME, 6-LANE FREEWAYS

FIG. NO.[a]	SITUATION DESCRIPTION, EQUATION, AND EQUATION VARIABLES[b]	MEAN	STD. DEVIATION	RANGE OF USE	NET REGRESSION COEFFICIENT		PARTIAL DETERMINATION COEFF.	LEVEL OF SIGNIFICANCE	R^2	STD. ERROR OF V_1	NO. OF LOCATIONS	NO. OF OBSERVATIONS
					VALUE	STD. ERROR						
8.9	*Standard on-ramp with adj. off-ramps*											
	$V_1 = -121 + 0.244V_f - 0.085V_u$								0.80		12	325
	$\qquad + 640V_d/D_d$											
	V_1 = volume in lane 1 (vph)	1041	312							140		
	V_f = freeway volume (vph)	4327	943	2400–6200	+0.244	0.008	0.72	0.01				
	V_r = on-ramp volume (vph)	594	349	100–1700								
	D_u = dist. to adj. upstrm. off-ramp (ft)	1459	558	900–2600								
	V_u = vol. of adj. upstrm. off-ramp (vph)	465	251	50–1100	−0.085	0.035	0.02	0.01				
	D_d = dist. to adj. dnstrm. off-ramp (ft)	2482	1404	900–5700								
	V_d = vol. of adj. dnstrm. off-ramp (vph)	449	260	50–1300								
	V_d/D_d			—[c]	+ 640	50	0.34	0.01				
8.10	*Upstream from off-ramp*											
	$V_1 = 94 + 0.231V_f + 0.473V_r$								0.84		11	277
	$\qquad + 215V_u/D_u$											
	V_1 = volume in lane 1 (vph)	1475	319							130		
	V_f = freeway volume (vph)	4590	1366	1100–6200	+0.231	0.007	0.79	0.01				
	V_r = off-ramp volume (vph)	556	365	20–1800	+0.473	0.030	0.47	0.01				
	D_u = dist. to adj. upstrm. on-ramp (ft)	2441	1716	900–5700								
	V_u = vol. of adj. upstrm. on-ramp (vph)	465	287	50–1200								
	V_u/D_u			—[d]	+215	60	0.05	0.01				
8.11	*Cloverleaf inner loop with aux. lane*											
	$V_1 = -87 + 0.225V_f - 0.140V_r$								0.64		7	136
	$\qquad + 0.500V_d$											
	V_1 = volume in lane 1 (vph)	878	293							178		
	V_f = freeway volume (vph)	3792	790	2000–5600	+0.225	0.019	0.51	0.01				
	V_r = on-ramp volume (vph)	810	383	200–1500	−0.140	0.040	0.08	0.01				

Ref	Variable											
	D_d = dist. to adj. dnstrm. off-ramp (ft)			400–850								
	V_d = vol. of adj. dnstrm. off-ramp (vph)	447	282	150–1500	+0.500	0.054	0.39	0.01				
8.12	*Diamond, etc., with aux. lane*								0.86		6	155
	$V_1 = 53 + 0.283V_f - 0.402D_d + 0.547V_d$									159		
	V_1 = volume in lane 1 (vph)	1164	419									
	V_f = freeway volume (vph)	4139	1179	1900–5600	+0.283	0.013	0.76	0.01				
	D_d = dist. to adj. dnstrm. off-ramp (ft)	627	222	300–1400	−0.402	0.079	0.15	0.01				
	V_d = vol. of adj. dnstrm. off-ramp (vph)	354	221	50–1000	+0.547	0.090	0.20	0.01				
8.13	*Second of successive on-ramps*								0.91		5	104
	$V_1 = 574 + 0.228V_f - 0.194V_r$ $- 0.714D_u + 0.274V_u$									142		
	V_1 = volume in lane 1 (vph)	686	453									
	V_f = freeway volume (vph)	3273	1005	1800–5400	+0.228	0.017	0.64	0.01				
	V_r = on-ramp volume (vph)	480	422	100–1500	−0.194	0.036	0.22	0.01				
	D_u = dist. to adj. upstrm. on-ramp (ft)	891	261	500–1100	−0.714	0.075	0.48	0.01				
	V_u = vol. of adj. upstrm. on-ramp (vph)	348	222	100–1400	+0.274	0.089	0.09	0.01				

[a] Corresponding nomograph in Chapter Eight, giving situation sketch, equation, conditions for use, limiting factors, and steps in use of equation and nomograph.
[b] All variables independent except V_1.   [c] See V_d and D_d.   [d] See V_u and D_u.

TABLE C.3—STATISTICAL BACKGROUND OF EQUATIONS FOR LANE 1 VOLUME, 8-LANE FREEWAYS

FIG. NO.[a]	SITUATION DESCRIPTION, EQUATION, AND EQUATION VARIABLES[b]	MEAN	STD. DEVIATION	RANGE OF USE	NET REGRESSION COEFFICIENT VALUE	NET REGRESSION COEFFICIENT STD. ERROR	PARTIAL DETERMINATION COEFF.	LEVEL OF SIGNIFICANCE	R^2	STD. ERROR OF V_1	NO. OF LOCATIONS	NO. OF OBSERVATIONS
8.14	*Standard on-ramp* $V_1 = -312 + 0.201V_f + 0.127V_r$								0.68		5	104
	V_1 = volume in lane 1 (vph)	974	270							156		
	V_f = freeway volume (vph)	5979	1072	3000–7700	+0.201	0.014	0.66	0.01				
	V_r = on-ramp volume, (vph)	833	220	300–1300	+0.127	0.071	0.03	0.10				
8.15	*On-ramp with dnstrm. off-ramp* $V_1 = -353 + 0.199V_f - 0.057V_r + 0.486V_d$								0.88		2	48
	V_1 = volume in lane 1 (vph)	851	292							107		
	V_f = freeway volume (vph)	5367	1149	3000–7100	+0.199	0.016	0.79	0.01				
	V_r = on-ramp volume (vph)	739	223	300–1100	−0.057	0.074	0.01	0.25				
	D_d = dist. to adj. dnstrm. off-ramp (ft)			1500–3000								
	V_d = vol. of dnstrm. off-ramp (vph)	368	160	100–800	+0.486	0.116	0.28	0.01				
8.16	*On-ramp with auxiliary lane* $V_1 = 584 + 0.180V_f - 0.203V_r - 0.487D_d + 0.204V_d$								0.86		5	72
	V_1 = volume in lane 1 (vph)	1237	274							107		
	V_f = freeway volume (vph)	5897	897	3670–7500	+0.180	0.015	0.68	0.01				
	V_r = on-ramp volume (vph)	515	255	110–1220	−0.203	0.058	0.15	0.01				
	D_d = dist. to adj. dnstrm. off-ramp (ft)	856	226	500–1100	−0.487	0.068	0.43	0.01				
	V_d = vol of dnstrm. off-ramp (vph)	557	279	110–1220	+0.204	0.051	0.20	0.01				

[a] Corresponding nomograph in Chapter Eight, giving situation sketch, equation, conditions for use, limiting factors, and steps in use of equation and nomograph.
[b] All variables independent except V_1.

TABLE C.4—STATISTICAL BACKGROUND OF EQUATIONS FOR 2-LANE RAMP AND MAJOR FORK VOLUMES, 6-LANE FREEWAYS

FIG. NO.[a]	SITUATION DESCRIPTION, EQUATION, AND EQUATION VARIABLES[b]	MEAN	STD. DEVIATION	RANGE OF USE	NET REGRESSION COEFFICIENT VALUE	NET REGRESSION COEFFICIENT STD. ERROR	PARTIAL DETERMINATION COEFF.	LEVEL OF SIGNIFICANCE	R^2	STD. ERROR OF V_1, V_{1+A} OR V_c	NO. OF LOCATIONS	NO. OF OBSERVATIONS
8.17	*2-Lane on-ramp with accel. lane*										2	115
	(a) Lane 1 calculation								0.23			
	$V_1 = 54 + 0.070V_f + 0.049V_r$									99		
	V_1 = volume in lane 1 (vph)	270	112									
	V_f = freeway volume (vph)	1625	611	600–3000	+0.070	0.016	0.15	0.01				
	V_r = on-ramp volume (vph)	2091	453	1100–3000	+0.049	0.021	0.04	0.02				
	(b) (Lane 1 + ramp lane A) calculation								0.74			
	$V_{1+A} = -205 + 0.287V_f + 0.575V_r$									212		
	V_{1+A} = vol. in lane 1 + ramp lane A (vph)	1464	413									
	V_f = freeway volume (vph)	1625	611	600–3000	+0.287	0.034	0.39	0.01				
	V_r = on-ramp volume (vph)	2091	453	1100–3000	+0.575	0.046	0.58	0.01				
8.18	*2-Lane off-ramp with decel. lane*										1	94
	(a) (Lane 1 + ramp lane A) calculation								0.68			
	$V_{1+A} = -158 + 0.035V_t + 0.567V_r$									181		
	V_{1+A} = vol. in lane 1 + ramp lane A (vph)	1093	315									
	V_t = freeway volume (vph)	3639	814	2100–6000	+0.035	0.026	0.02	0.20				
	V_r = off-ramp volume (vph)	1983	433	1100–3000	+0.567	0.049	0.59	0.01				
	(b) Lane 1 calculation								0.18			
	$V_1 = 18 + 0.060V_t + 0.072V_r$									150		
	V_1 = vol. in lane 1 (vph)	378	163									
	V_t = freeway volume (vph)	3639	814	2100–6000	+0.060	0.022	0.08	0.01				
	V_r = off-ramp volume (vph)	1983	433	1100–3000	+0.072	0.041	0.03	0.10				
8.19	*Major fork*										3	84
	(a) (Lane 1 + lane A) calculation								0.91			
	$V_c = 64 + 0.285V_t + 0.141V_r$									94		
	V_c = vol. in lane 1 + ramp lane A (vph)	1151	314									
	V_t = freeway volume	3170	847	1200–4500	+0.285	0.016	0.80	0.01				
	V_r = off-ramp[c] volume (vph)	1290	571	300–2650	+0.141	0.024	0.31	0.01				
	(b) Lane 1 calculation								0.90			
	$V_1 = 173 + 0.295V_t - 0.320V_r$									66		
	V_1 = volume in lane 1 (vph)	694	201									
	V_t = freeway volume (vph)	3170	847	1200–4500	+0.295	0.011	0.90	0.01				
	V_r = off-ramp volume (vph)	1290	571	300–2650	-0.320	0.017	0.82	0.01				

[a] Corresponding nomograph in Chapter Eight, giving situation sketch, equation, conditions for use, limiting factors, and steps in use of equation and nomograph.
[b] All variables independent except V_1, V_{1+A}, and V_c.   [c] "Off" leg at major fork.

APPENDIX D

AVERAGE VOLUME DISTRIBUTIONS,
BY LANE, UPSTREAM OF
ON-RAMP JUNCTIONS

Prior to the development of the detailed series of equations and nomographs presented in Chapter Eight for lane 1 service volumes upstream of on-ramp junctions in levels A through C, more general volume distribution criteria had been developed as an interim measure (1, in Ch. 8). These consisted of a series of curves presenting average volume distributions by lane upstream of on-ramp junctions, for a variety of geometric conditions.

These freeway volume distributions by lanes are given in Figures D.1, D.2, and D.3. They are taken just upstream from the ramp nose before the merge has taken place.

For 4-lane freeways, the freeway volume distributions are presented in Figure D.1 in two groups—those at cloverleaf inner loop on-ramps and those at all other types of on-ramps. The reason for this grouping is the difference in operation at cloverleaf interchanges caused by traffic weaving between

the adjacent inner loops. In comparison with other types of ramps, the cloverleaf inner loop ramp curves show a heavier use of lane 1 up to freeway volumes of 2,400 vph, despite the loss of lane 1 vehicles at the upstream adjacent outer connection off-ramp. Much of the lane 1 traffic is destined for the downstream inner loop off-ramp only 400 to 700 ft away. At freeway volumes above 2,400 vph the comparison shows a heavier use of lane 2 at cloverleaf locations, possibly because drivers wish to avoid the more severe merging and weaving conflicts present at high-volume cloverleaf interchanges.

Figure D.2 is derived from 6-lane freeway volume distributions at all types of on-ramps, other than cloverleaf inner loops, where no auxiliary lane is present between the on-ramp and the adjacent downstream off-ramp.

Similar curves for the "with auxiliary lane" and the cloverleaf cases are not included because few generalized data are
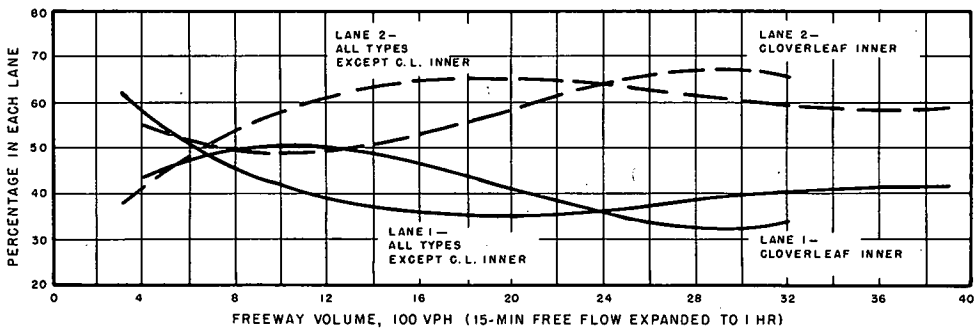

Figure D.1. Volume distribution on 4-lane freeways upstream from cloverleaf inner loop and from all other types of on-ramps.
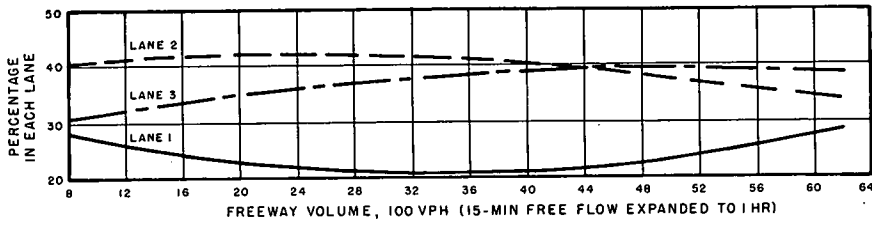
396

Figure D.2. *Volume distribution on 6-lane freeways upstream from all types of on-ramps (without auxiliary lane at on-ramp entrance) except cloverleaf inner loops.*
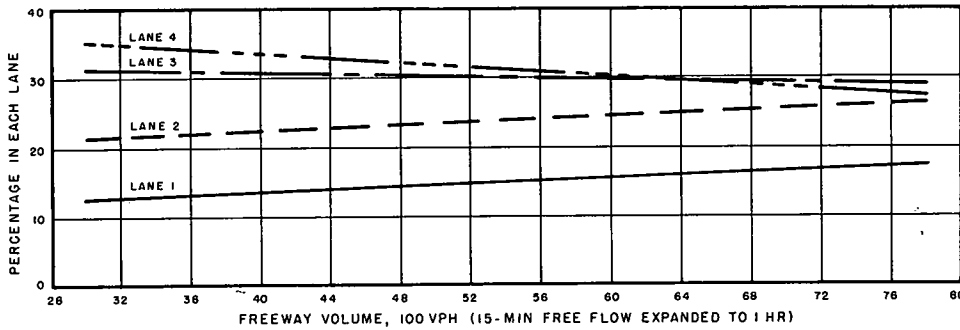

Figure D.3. *Volume distribution on 8-lane freeways upstream from on-ramps.*

available separate from the information on which the equations and nomographs are based.

Figure D.3 provides approximate data for all types of on-ramp locations on 8-lane freeways.

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Under the terms of its Congressional charter, the Academy is also called upon to act as an official—yet independent—adviser to the Federal Government in any matter of science and technology. This provision accounts for the close ties that have always existed between the Academy and the Government, although the Academy is not a governmental agency and its activities are not limited to those on behalf of the Government.

The NATIONAL ACADEMY OF ENGINEERING was established on December 5, 1964. On that date the Council of the National Academy of Sciences, under the authority of its Act of Incorporation, adopted Articles of Organization bringing the National Academy of Engineering into being, independent and autonomous in its organization and the election of its members, and closely coordinated with the National Academy of Sciences in its advisory activities. The two Academies join in the furtherance of science and engineering and share the responsibility of advising the Federal Government, upon request, on any subject of science or technology.

The NATIONAL RESEARCH COUNCIL was organized as an agency of the National Academy of Sciences in 1916, at the request of President Wilson, to enable the broad community of U.S. scientists and engineers to associate their efforts with the limited membership of the Academy in service to science and the nation. Its members, who receive their appointments from the President of the National Academy of Sciences, are drawn from academic, industrial and government organizations throughout the country. The National Research Council serves both Academies in the discharge of their responsibilities.

Supported by private and public contributions, grants, and contracts, and voluntary contributions of time and effort by several thousand of the nation's leading scientists and engineers, the Academies and their Research Council thus work to serve the national interest, to foster the sound development of science and engineering, and to promote their effective application for the benefit of society.

The DIVISION OF ENGINEERING AND INDUSTRIAL RESEARCH is one of the eight major Divisions into which the National Research Council is organized for the conduct of its work. Its membership includes representatives of the nation's leading technical societies as well as a number of members-at-large. Its Chairman is appointed by the Council of the Academy of Sciences upon nomination by the Council of the Academy of Engineering.

The HIGHWAY RESEARCH BOARD, organized November 11, 1920, as an agency of the Division of Engineering and Industrial Research, is a cooperative organization of the highway technologists of America operating under the auspices of the Academy-Council and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of highway transportation. The purposes of the Board are to encourage research and to provide a national clearinghouse and correlation service for research activities and information on highway administration and technology.